THE UNIVERSITY OF ALBERTA

EXPERIMENTAL CONCRETE PAVEMENTS IN ALBERTA

A DISSERTATION

SUBMITTED TO THE FACULTY OF GRADUATE STUDIES

IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE

OF MASTER OF SCIENCE

DEPARTMENT OF CIVIL ENGINEERING

by

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EDMONTON, ALBERTA
April 4th, 1960.

ABSTRACT

This thesis gives the first account of the structural performance of experimental concrete pavements constructed near Calgary. These experimental concrete roads were planned to discover the suitability of concrete construction for Alberta conditions, using designs, in most cases, proven elsewhere.

The types of pavement examined were:

Type A: 6 inch, 7 inch and 8 inch slabs reinforced at 8 lbs/square yard, with sawn doweled joints at 30 feet (constructed 1958 and 1959)

Type B: 6 inch and 7 inch slabs continuously reinforced with 0.7 per cent to 0.8 per cent steel (constructed 1958)

Type C: 8 inch plain concrete slab with sawn doweled joints at 20 feet (constructed 1958)

Type D 8 inch plain concrete slab with doweless joints at 20 feet (constructed 1955)

Except in special cases, no expansion joints were used, and each pavement type was constructed with a granular base.

The author's analysis has been made principally by observations of such features as cracking behaviour and joint condition. A detailed examination of joint width variations has been made, to investigate the behaviour of pavements with the contraction joint design.

At the end of one year the Type A pavement of 6 inch slabs was performing as satisfactorily as the heavier Type A pavements. The pavements with unreinforced slabs appear to be deteriorating more quickly because of wider and more frequent uncontrolled cracks. However, most of the uncontrolled cracks in the plain concrete slabs cannot be attributed entirely to loading, but perhaps to unfortunate construction techniques and excessive dowel resistance. If the criterion for performance is the rate of cracking, the performance of the plain concrete with no dowels is satisfactory, while

the plain concrete with dowels is worse than might be expected.

Faulting is starting to occur on the type D pavement, and also at uncontrolled cracks in the plain concrete generally.

Hence, with conditions as they are at present on these test roads, it can be concluded that a 6 inch slab with reinforcement and dowels appears superior to an 8 inch plain slab with no reinforcement, with or without dowels. The indications are that a plain slab without dowels is preferable to one with dowels.

No direct pumping has yet been observed on these pavements, which all have at least 4 inches of base, but water has been seen to ooze from joints, in particular near the bottom of a 5 per cent grade.

From the joint measurements, typical values for the coefficient of linear expansion, shrinkage strain and creep strain in the concrete have been obtained.

As a future trial it is suggested that the conventionally reinforced slab length should be increased to 100 feet. Doweled joints with high load transfer, but low frictional resistance, should be used.

Field observations on the continuous pavements have been mainly to record the cracking patterns and the development of cracking. It is suggested that the possibility of inducing crack spacing at some desired uniform interval should be investigated. The maximum steel stress in such a pavement might then be less than in a similar pavement allowed to crack naturally.

At present, it can be concluded that the lightest continuous slab, i.e. 6 inches thick, with 0.72 per cent steel, is performing as satisfactorily as the heavier and thicker slabs.

Riding quality was measured by a simple HiLo Roughometer, manufactured by "Soiltest". This method seems to be satisfactory for comparing the riding quality and deterioration of riding quality of different types of construction. But the results cannot be directly compared with results from elsewhere by different methods.

The final assessment of performance of these experimental concrete sections, and a comparison with adjacent bituminous pavements, cannot be made until significant failures develop. Significant failures are not likely to develop for several years.

ACKNOWLEDGEMENTS

A student, thoughout his University career, is stimulated and influenced by all who lecture to him and direct his studies. Thus, he is indebted to a great many people.

But in the case of the author, however, a post-graduate University career would not have been possible at all without the assistance of a Canadian Good Roads Association scholarship, provided through the generosity of the Standard Gravel Company, of Calgary.

Also, for his research project, the author is indebted to the sponsorship of the Alberta Department of Highways, through the Alberta Research Council, and in particular to the assistance of Mr. W.E.Curtis, the Highways Materials Engineer, and Mr.P.Fetsko, Resident Engineer in charge of construction of the project.

The author was responsible to Mr. B.P.Shields, Head, Highways Section, Alberta Research Council, during the summer, when field work was undertaken. The author wishes to record thanks for the many suggestions that Mr. Shields made throughout the year, but in particular when the author was drafting his thesis. In addition, the willing assistance of Alberta Research Council technicians D.Shepperd and F.Vaneldick must be mentioned for the numerous all-night and very early morning measuring sessions which had to be taken to fit in with the idiosyncrasies of the pavement behaviour.

Finally, the author is indebted in particular to his supervisor, Professor S.R.Sinclair, and also to Assistant Professor K.O.Anderson, who willingly allowed the author to draw from his copious supplies of literature.

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CHAPTER I INTRODUCTION

Considerable literature is available concerning concrete pavements, and most of this information appears in the form of reports on test sections of concrete roads, or summaries and conclusions from such reports. From the backlog of experience that is available in this literature it is now possible to design and specify concrete road structures of adequate durability.

Concrete pavements that have given satisfactory performance in other areas can be broadly classified into the following groups:

1. Pavement slabs without joints, but with continuous reinforcing.
2. Pavement slabs with contraction joints. These pavements can be further classified into:
   a. Unreinforced slabs
   b. Lightly reinforced slabs

In the U.S.A., jointed pavements are generally constructed with slab thicknesses between eight inches and ten inches, but it is suspected that continuous pavements are durable with an inch or two less. A compacted granular base and a uniformly compacted subgrade are now considered necessary for satisfactory performance.

Several examples of each type of concrete pavement embodying the features mentioned above have been constructed as test sections on two heavily trafficked highways near Calgary, southern Alberta. Also, three sections have been laid down with a slab thickness of six inches, which appears to be less than would be used in the U.S.A. for this class of road.*

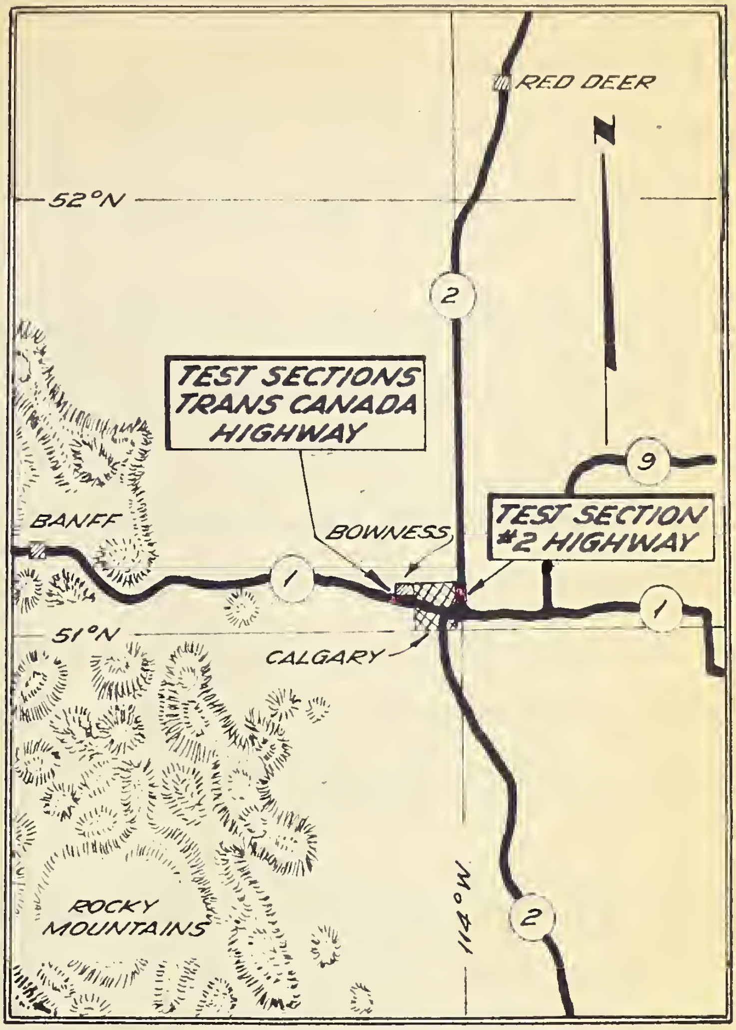
* Portland Cement Association "Charted Summary of Concrete Road Specifications Used by State Highway Departments, 1958" Also, see Appendix D, which outlines the design that would be chosen from the A.C.I. Committee 325 Recommendations.

The purpose of these Alberta test roads is not to examine radical new ideas of design. But rather it is to observe the manner in which we can expect concrete highways of proven design in the U.S.A. to react to the Alberta climate and traffic conditions. The ultimate aim is to compare the behaviour of these concrete pavements with heavy-duty asphaltic concrete pavements which have been constructed adjacent to the concrete test sections. However, the study for this Thesis has been limited to the concrete pavements alone, and the behaviour of the asphaltic pavements has not been considered.

In 1955, the first experimental road, one mile long, was constructed on #2 Highway, just north of Calgary. This was a new location, and included also heavy duty asphaltic concrete pavement.

The second experimental road, five miles long, was constructed on #1 Highway, which is part of the Trans-Canada Highway. The location is near the town of Bowness, just west of Calgary. Construction was started in 1958 and completed in 1959.

A plan of these locations is given on plate 1.



SCALE 30 MILES TO 1 INCH APPROX.

CLIMATIC DATA:

AVERAGE JANUARY TEMP. 16°F.
LOWEST JANUARY TEMP. -46°F.
AVERAGE JULY TEMP. 62°F.
HIGHEST JULY TEMP. 97°F.
AVERAGE ANNUAL PRECIPITATION 17½"

PLATE 1

CHAPTER II SUPERVISION AND RESEARCH

Survey and soils data was collected by the Alberta Department of Highways, who drew up the construction drawings. Both test sections were constructed under Contract by Wells Construction Limited, of Calgary. The Alberta Department of Highways was responsible for the supervision of the Contract and routine testing.

The research engineering and instrumentation has been carried out by the Alberta Research Council. Two engineers, C.Ellert and J. Berznicki, were employed in 1958 by the Research Council to analyse the performance of the pavements. Berznicki examined the stresses in the steel, and Ellert considered principally joint width variations, cracking patterns, and pavement roughness. In 1959, the research of Berznicki was continued by W. Babowal, and the research of Ellert by the writer.

Thus, the research of the writer is in the nature of a first report on the general behaviour of these experimental roads, and is to compare the performance of the different types of concrete construction.

Information on the behaviour of contraction joints is also presented.

CHAPTER III DESIGN AND CONSTRUCTION DETAILS

Plates 3A, 3B and 3C summarize the features of the design and construction, which are detailed below. Construction drawings from the Contract are given in Appendix A.

Geometric Details

The cross section of the Test Sections on #1 Highway, the Trans-Canada Highway, consists of two 12 foot slabs constructed monolithically, and bordered by ten foot wide hot plant mix, bituminous surfaced, shoulders.

The maximum gradient is five per cent, and the sharpest horizontal curvature $2\frac{1}{2}$ degrees. This Highway is a two-lane highway, with the shoulders for emergencies or parking.

The #2 Highway Test Sections were constructed with four separate twelve foot slabs bordered by eight foot wide MC2 asphalt bound gravel shoulders. The gradients are in the order of one per cent maximum, and there is no horizontal curvature. This Highway is a four-lane highway, with the shoulders for emergencies or parking.

Thicknesses of Construction

#1 Highway	Type A	30 ft. Conventionally Reinforced Slabs (steel at 8 lbs/sq.yard with doweled contraction joints)
	A-8	8 inch Slab 4 inch Base (1.22 miles)
	A-7	7 inch Slab 4 inch Base (0.59 miles)
	A-6	6 inch Slab 8 inch Base (0.49 miles)
	Type B	Continuously Reinforced Slabs
	B-7-1	7 inch Slab 4 inch Base 0.71 per cent steel (0.50 miles)

	B-7-2	7 inch Slab 4 inch Base 0.78 per cent steel (0.51 miles)
	B-6-1	6 inch Slab 4 inch Base 0.72 per cent steel (0.37 miles)
	B-6-2	6 inch Slab 4 inch Base 0.82 per cent steel (0.42 miles)
	Type C	20 foot Plain Concrete Slabs (with doweled contraction joints) 8 inch Slab 6 inch Base (0.98 miles)
#2 Highway	Type D	20 foot Plain Concrete Slabs (with no dowels or tie-bars) 8 inch Slab with outer foot of each 12 foot lane, increased to 9" to allow for poor compaction of base adjacent to form.

Subgrade

The subgrade soil type on both projects was reasonably uniform throughout, and according to the Casagrande classification soils would both be type CL. The subgrade soil on #1 Highway is a medium plastic silty clay, and on #2 Highway a silty or sandy clay.

Atterberg Limits

#1 Highway Liquid Limit per cent

Maximum 39	Minimum 18	Average 24

Plasticity Index percent

Maximum 23	Minimum 0	Average 10

#2 Highway Liquid Limit per cent

Maximum 29	Minimum 22	Average 25

Plasticity Index per cent

Maximum 16	Minimum 4	Average 10

Compaction of the Grade

The top twelve inches were specified to be 100 per cent Standard Proctor density and lower in the grade to be 95 per cent Standard Proctor density. Construction of the pavements was started soon after the grade had been completed.

Soils Testing on #1 Highway

Field densities were taken to check the specification limits. In addition, field California Bearing Ratio (CBR) tests, unconfined compression tests, and moisture contents were taken to determine the properties of the subgrade. The modulus of subgrade reaction or "k" value, was estimated from laboratory CBR tests at 100 per cent Standard Proctor density, allowing for the spring loss of strength (Motl Committee Report). The field CBR was 12 per cent to 35 per cent and the estimated "k" value 170 pounds per cubic inch.

Soils Testing on #2 Highway

Similar tests were taken, except that the strength tests were in less detail. Field CBR values of between 12 per cent and 35 per cent (corrected for spring loss of strength) were obtained.

Frost Susceptibility

From the soils data the Department of Highways estimated that the subgrade on #1 Highway had medium to high frost potential, and on #2 Highway the subgrade was reasonably non-frost-susceptible.

Base on #1 Highway

The base on both projects was classified as "D.O.H. #1 Class A 3/4 inch Crush", this being considered good quality material.

The grading limits and a typical result are given below. All material for the base in the concrete test section on #1 Highway came from the same pit and was uniformly graded.

Table 3-1. Base Course Grading

Sieve Size	Spec.Limit % Passing	Typical Actual Grading	Remarks
3/4 inch	100	100	
#4	40 - 60	49	MacIntosh Pit
#10	25 - 45	37	May 2nd, 1959
#40	10 - 25	23	Station 249+89 4' L
#200	2 --10	9	Standard Proctor 132.2
PI	Not greater		lbs/ft^3
	than 10%		Actual Moisture 2.7%

Frequent tests were carried out to ensure that the base fell within the specifications. The base was compacted to 100 per cent Standard Proctor density and checked by field tests.

Base on #2 Highway

This consisted of five inches of two inch crushed gravel with a one inch dressing of 3/4 inch crushed gravel. Compaction was to 100 per cent Standard Proctor density.

Concrete Mix

The mixes were designed to have a flexural strength of 550 p.s.i. at ten days, with a slump of one to two inches, using three per cent to six per cent entrained air. The actual mix designs used are given in the table below:

Table 3-2 Concrete Mix Design

Date	Approx. Stations Commencing	Cement	Quantities lbs/yd.³ Sand	1½" Agg.	3/4" Agg.	Additive
#1 Highway						
1. Sep.4 1958	425 + 75	545	1210	1400	700	WRDA
2. Oct.17 1958	295 + 00	600	1190	1165	770	Darex
3. Apr.27 1959	253 + 00	610	1240	1110	750	"
4. May 11 1959	245 + 00	610	1190	1165	770	"
#2 Highway						
June 1955		590	1135	1290	695	

Water added to these mixes was between 17 and 19½ galls/yd.

Aggregates

The grading of the aggregates was governed by the A.S.T.M. specifications for Concrete Aggregates. #1 Highway was a 1½ inch grading and #2 Highway a 2 inch grading.

The specification for deleterious materials was as follows:

Material passing #200 sieve -	not greater than 3 per cent
Coal	1 per cent
Clay lumps	1 per cent
Shale	1 per cent
Other deleterious material	1 per cent

The total quantity of deleterious material could not exceed 3 per cent.

Batching and Mixing

On #1 Highway the mix was batched by weight from stockpiles

of three different sizes and transported to the mixer in trucks with compartments for three 1 cub. yard batches.

Water and air entraining agent were added at the paver mixer, adjustments being made to obtain the correct slump and air. The mix was dropped between the forms by a 1 cub. yard clamshell bucket on a long boom. The reinforcement was placed before any concreting took place.

Spreading, Compacting and Finishing

The spreader was of the reciprocating blade type, and included a vibrating beam compactor.

The finishing machine carried a transverse screed and a transverse float, and was followed by a 12 foot longitudinal float.

The process was completed by mechanical brooming with a steel bristle broom, and application of the curing compound by a spraying machine. The desired appearance of the surface after construction is illustrated by plate 3-D.

Similar methods of construction were used on #2 Highway, except that the surface was roughened by a burlap drag, and curing carried out by means of dampened burlap.

Concrete Tests

The grading specifications were controlled by sieve analyses from the various stockpiles. A typical result is given below in table 3-3.

Table 3-3. Grading Analysis of the Concrete Aggregates

Sieve Size	Fine Agg. % Passing	Medium Agg. % Passing	Coarse Agg. % Passing
1½"			100
1"			46.6
3/4"		100	14.6
3/8"	100	53.9	1.3
#4	97.4	9.6	
#8	74.2	2.2	
#16	50.0		
#30	38.6		
#50	18.9	% Crush 65%	% Crush 60%
#100	6.9	Mostly angular -- some round	
#200	2.2		
Finess Mod	3.1		

Slump and air content were controlled by samples taken at frequent intervals and tested on the site. A pressure type air indicator was used.

On #1 Highway, flexural strength tests were carried out in accordance with American Society for Testing Materials (A.S.T.M.) recommendation C-78. The broken beams from these tests were tried for compressive strength in accordance with A.S.T.M. C-116. In addition, cylinder tests for compressive strength were made.

For flexural strength, and the C-116 compressive strength, two samples were tested at ten days, and one at twenty-eight days. For the cylinders, two tests were made at twenty-eight days.

Strength, slump, and air content data, supplied by the Department of Highways, are given in Appendix B.

Curing

On #1 Highway, a resinous membrane curing compound was sprayed on by a machine, set to give uniform coverage. Both white and pigmented membranes were used, generally at about twice the Manufacturer's recommended rate.

On #2 Highway the slabs were cured by wet burlap.

Cement Tests

The cement was supplied by the Canada Cement Company. Tests carried out were for Time of Set (Vicat method) and Normal Consistency.

On #1 Highway the initial set ranged between 1½ and 2½ hours, and the final set between 3½ and 4½ hours. The normal consistency was close to 24 per cent.

Transverse Joint Spacing

For the plain concrete, types C and D, the joints were spaced at 20 foot intervals, while on type A, joints were at 30 foot intervals. The type B contained no deliberate joints, and the steel was carried straight through the construction joints at the end of each day's work.

Transverse Joint Details

On pavements A and C, the joints were of the contraction type, with a saw-cut to quarter depth to induce cracking. #8 x 18 inch hard steel dowels, graphite rubbed in the leading slab, were used on 15 inch centres.

On the type C, these dowels were held by wire chairs, supporting each end of the dowel and carried through on the surface of the base.

On the type A pavement, the dowels were tied to the reinforcing bars. (See plate 3-D.)

Only four expansion joints were used in the pavements A and C, and these were 30 feet either side from the bridge abutments in type A-8 and 30 feet from the expansion dams in types C and A-6.

The expansion dams separted the type B pavements from the remainder, and consisted of interlocking steel tongues on the surface of the road slab. The road slab was supported from beneath at the dam by a concrete base slab set into the gravel base. The initial clear opening in the dam at the time of construction was three inches.

On the type D there were no dowels used. The joints were sawn to quarter depth except that every third joint was formed right through by a steel plate.

Longitudinal Joints

The centre line joint on #1 Highway was of the dummy type, sawn to quarter depth with #4 deformed tie bars at 30 inch centres. On type D there were three ongitudinal joints each formed with a 2 inch by 4 inch key way; no tie bars were used.

Sawing Procedure

The transverse joints were first sawn a few feet in from one edge. The saw was then removed to the other edge and the cut completed.

Transverse joints were generally sawn during the day following their construction. On pavement type A-7, and some of A-8, the transverse joints were sawn consecutively. On other parts of A-8 alternate transverse joints were first sawn, followed by sawing the intermediate joints. No "control" joints at wide spawings were sawn.

The centre line joint was sawn at any convenient time since it was not imperative to saw the day after construction.

<u>Reinforcement</u>

The reinforcement consisted of intermediate grade deformed steel bars.

Type A. Longitudinal #4 deformed bars @ 22" c.c. with an extra bar at closer spacing near the edge.

Transverse #4 deformed bars @ 23" c.c.

Total weight 8 lbs/sq.yard

Type B. The quantity of longitudinal steel was varied to form four different types of pavement.

Longitudinal B-7-(1) #8 deformed bars @ 16" c.c. (% steel 0.71)

(2) #8 deformed bars @ 15" c.c. (% steel 0.78)

B-6-(1) #6 deformed bars @ 10" c.c. (% steel 0.72)

(2) #6 deformed bars @ 9" c.c. (% steel 0.82)

Transverse #4 deformed bars @ 30" c.c.

Types C and D have no reinforcement.

For the type B slabs the longitudinal bars were placed and tied to 2½ inch risers. The transverse bars were laid on top and tied with wire wherever they crossed the longitudinal bars.

For the type A pavements the procedure was similar except that the reinforcement was joined together in the yard to make 12 foot x 30 foot bar mats.

Strength Tests on the Reinforcement

Tension tests carried out by Bereznicki* showed that the yield stress of the steel was 53,000 p.s.i. and the ultimate stress between 81,000 and 94,000 p.s.i. The strain at yield point was 1,800 micro inches per inch, and the ultimate strain was between 182,000 and 204,000 micro inches per inch.

Dates of Construction

Type D was constructed in June 1955. Types C, B and A-6 were constructed between September 8th and October 29th, 1958 and the project on Highway #1 was completed by constructing types A-7 and A-8 in May 1959.

* "Direct Measurement of Strain in Concrete Pavement Steel" page 195. Unpublished M.Sc Thesis, University of Alberta, 1959.

TRANS CANADA HIGHWAY TEST SECTIONS AT BOWNESS
LENGTH 5.08 MILES

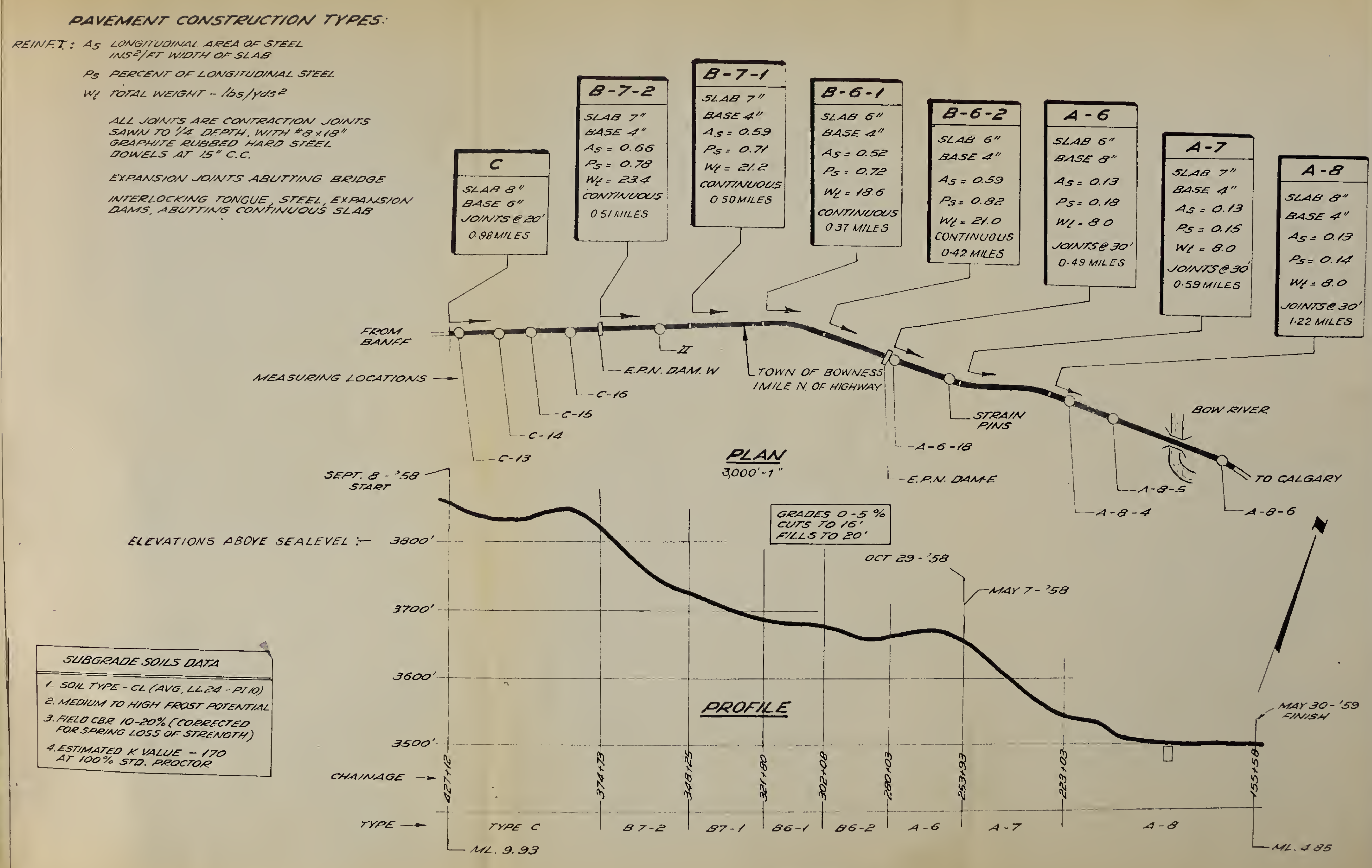
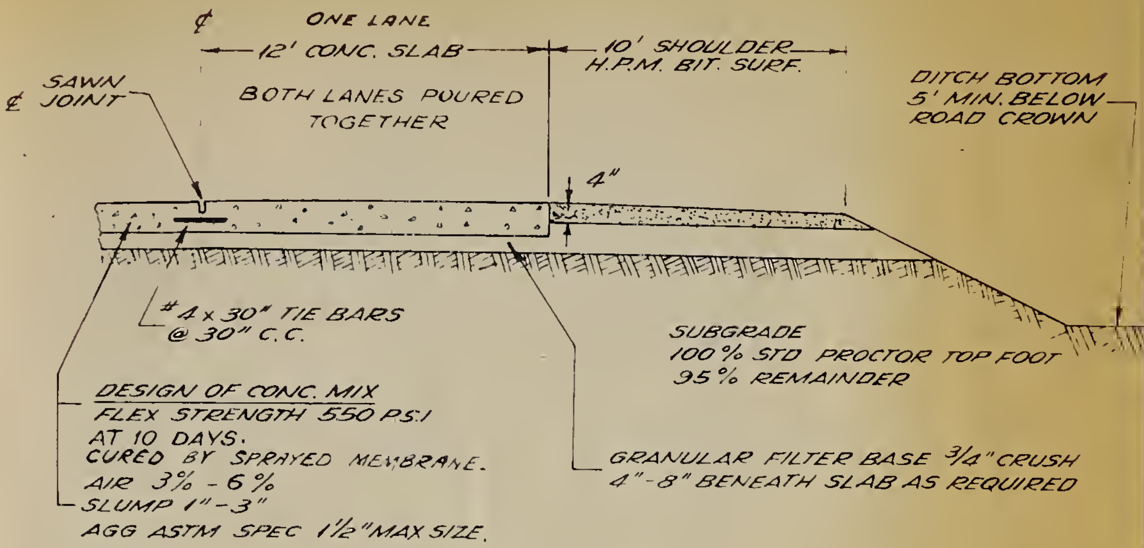


PLATE 3A

TYPICAL CROSS SECTION FOR TRANS CANADA HIGHWAY
(REINFORCEMENT OMITTED)



HALF CROSS-SECTION

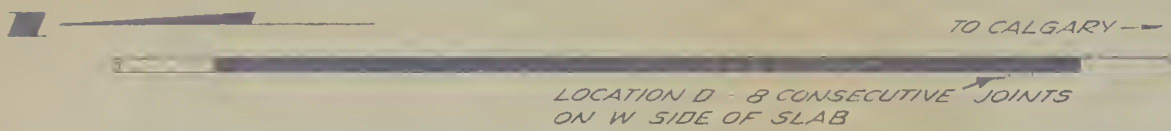
NEAR E. END OF PROJECT LOOKING EAST (LOCATION A-8 6)

PLATE 3-B

TEST SECTION "D" - # 2 HIGHWAY

LENGTH 0.91 MILES

PLAN 1000' = 1"

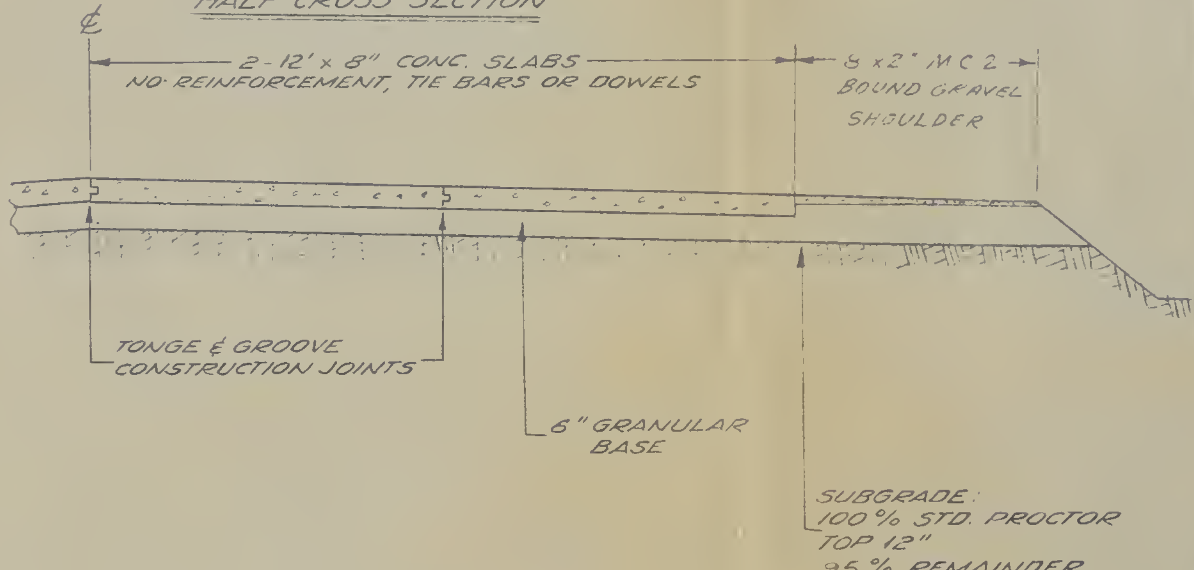


CONTRACTION JOINTS @ 20' EVERY THIRD JOINT FORMED REMAINDER SAWN

GRADE 0 - %

NO APPRECIABLE CUT OR FILL

HALF CROSS SECTION



CONC MIX DESIGN

FLEX STRENGTH 550 PSI @ 10 DAYS

AGG ASTM SPEC 2" DOWN

AIR 4% 6% SLUMP 1" - 3"

[illegible]

SUBGRADE SOIL DATA

1. TYPE CL (AVG LL 25 PI 10)
2. REASONBLY NON FROST SUSCEPTIBLE
3. FIELD CBR 12 % - 35 % (CORRECTED FOR SPRING LOSS OF STRENGTH

PLATE 3-[illegible]

SURFACE APPEARANCE FROM
MECHANICAL BROOM

FIXING THE DOWELS
TYPE A-8

PLATE 3-D

CHAPTER IV TRAFFIC

#1 Highway has a maximum speed limit of 60 m.p.h. for cars and 50 m.p.h. for trucks. The maximum allowed axle load is 18,000 lbs. for single axles, or 16,000 lbs. for each axle of a tandem axle. A gross load of 72,000 lbs. per vehicle is at present allowed.

#2 Highway has a maximum speed limit of 65 m.p.h. for cars, 55 m.p.h. for trucks, and the gross load is 62,000 lbs. per vehicle. The maximum axle loads are the same as on #1 Highway.

To date, the Authorities have felt a spring load ban unnecessary on either of the highways.

No records are available of traffic frequency, and it is suggested that, in particular, the rate and frequency of heavy trucks should in future be recorded.

CHAPTER V ASSESSMENT OF PERFORMANCE

To analyse the performance is a long-term process and consists of periodical examinations to locate "signs of distress". The progress of these signs of distress is recorded until individual failures occur; an individual failure could, for example, be a dangerous bump or depression. Complete or over-all failure of the highway will be due to the cumulative effect of individual failures.

The function of a highway is to carry vehicles with safety at high speeds, and hence the assessment of structural performance should be directed with this thought in mind. Visual observations of cracking, faulting, and the like, should be correlated with riding quality measurements. The riding quality should also be judged by qualified observers from standard vehicles.

Causes of Distress

Distress can be caused by deficiencies in the design of the pavement or the manner in which it was constructed, traffic loading and warping stresses, and slow deterioration through long-term effects of weather.

Distress from Causes related to Construction Procedures

Only the factors that seem relevant to these Alberta test roads are mentioned.

a. Surface deterioration due to excessive hand forming.

b. Shrinkage cracks due to improper curing techniques.

c. Spalling around sawn joints which can occur during sawing when the concrete is too green.

d. Transverse contraction cracks which, although caused by shrinkage, only occur if doweled joints remain frozen or the

slabs are too long.

e. Transverse cracks sympathetic with joints in cured adjacent slabs. The thermal contraction in the adjacent slabs sometimes causes cracking in the freshly poured adjacent concrete before its joints can be sawn.

f. Transverse cracks occurring because the slab is not sawn in time.

The features listed above can be minimized by experience and careful control during construction.

Subsequent Signs of Distress

The appearance of the features listed below depends upon the adequacy of the design of the pavement. Considerations in design should include the subgrade strength, the weight and frequency of traffic and climatic effects. The rate of development of the defects listed below governs the life of the pavement and they cannot always be eliminated even with the most careful design and construction procedures.

g. Transverse cracks due to warping. These are found in the central portion of the slabs and can occur with or without traffic loading.*

h. Cracks due to traffic loading. These normally start to occur following pumping, and are usually first found about five feet from the joint in the forward slab. Corner cracking and the general breakdown of the slab into smaller units frequently follows.+

* E.Yoder "Principles of Pavement Design" page 75, 1959

\+ F.Hveem "Types and Causes of Failure in Highway Pavements" H.R.B. Bulletin 187, figures 47 to 50, 1958.

Excessive overloading will cause fairly rapid overall failure, but normal maximum loading results in very gradual individual failures which accumulate until an overall condition of failure is reached.

i. Cracks due to deep seated settlement. These can usually be picked out by the appearance of broad undulations in the surface of the pavement.

j. Faulting due to excessive loading or loss of load transfer at the joint, sometimes aided by pumping. Faulting results in a rough riding surface.

k. Restraint cracking due to foreign material within the joint, which can also result from pumping, or seized and rusted dowels.

l. Spalling and crushing of the concrete around a joint, and blow-ups, due to expanding aggregates, foreign matter in the joints, or seized and corroded dowels.

Method of Assessment

It should be noted that many of the factors listed above will not occur in continuous pavements, and it is therefore convenient to consider the continuous pavements after the jointed pavements have bean dealt with.

Thus, the method of assessment used in this thesis will be

1. Consideration of the uncontrolled cracks in the jointed pavements (Chapter VI)
2. Pavement surface condition (Chapter VII)
3. Visual Assessment of joint behaviour in the jointed pavements (Chapter VIII)
4. Riding quality measurements (Chapter IX)
5. Cracking in the continuous slabs (Chapter XIII)

CHAPTER VI UNCONTROLLED CRACKS IN PAVEMENTS WITH JOINTS

Types of Cracking

At the time of the last surveys, all the major cracks were transverse cracks or shrinkage cracks, and no corner cracks, restraint cracks or longitudinal cracks were found. The cracks observed are classified below and a discussion of each type is given. A plan of the uncontrolled cracks in types C and D is given on plate 6-A

Classification of the Cracks

1. Close to joints which are thought to have seized dowels. These uncontrolled cracks formed during the initial shrinkage period, and can be described as "wide" cracks. On the reinforced slabs they tended to run roughly parallel to the joint right across the slab (see plate 6-B), while on the type C they frequently describe a rough arc from some point along the joint to the edge of the slab, several feet away from the joint.

2. Transverse cracks also caused by seized dowels, but occurring several feet from the joints. These cracks were noticed before the road was opened to traffic.

3. Transverse cracks that did not develop immediately but started after the initial shrinkage period, and are still developing to date. From the description previously given in Chapter V (items h and g), it might be supposed that they are caused by loading or warping. However, on this project stiff joints combined with subgrade frictional forces might exceed even the tensile strength of the matured concrete and be the major cause for this type of crack. It is felt that they are not the result of excessive settlement.

4. Cracks and spalls that occurred around a sawn joint during

sawing. An example is given in plate 6-C, figure 2, and probably occurred when the concrete was sawn too green. The crack shown in plate 6-C, figure 1, has also been classified in this group although it is not known exactly how or when it occurred, but it is suspected that it occurred during sawing. Three joints on the type C pavement show this latter form of cracking.

5. Shrinkage cracks. These were numerous, particularly on the type C, and occurred when the application of the curing compound was delayed. Examples are given on plates 6-C and 6-D. Further discussion of shrinkage cracks will be given in due course.

6. Sympathetic cracks. These were exactly as described under (e) in Chapter V and occurred on the type D pavements.(See plate 6B)

7. Uncontrolled cracks occurring because sawing of the slab was too long delayed. These also mainly occurred on the type D pavements. Many of the cracks on the type D pavements are in very bad condition (see plate 6-D).

Remark

It is interesting to note that the type 1, as listed above, has been observed in Manitoba. R.N.Sharpe* makes the following statement: "Observations on some of our early post-war concrete pavements show that a number of slabs were cracking approximately one foot back from transverse joints. After considerable study it was concluded that these cracks were due in part to misalignment of the dowel assembly, and in part to excess of bond in the sliding end of the dowels".

On the following page in Table 6-1, a summary of the uncontrolled cracks is given.

* "Design and Construction of Concrete Pavements in Manitoba" Proceedings C.G.R.A. Convention 1957.

TABLE 6-1 Summary of Major Uncontrolled Cracks in the Jointed Pavements

Pavement Type	Length Miles	Constructed	Last Survey	Type 1	Type 2	Type 3	Type 4	Type 6	Type 7	Faulting
A-6	0.49	Oct.1958	Jan.1960			3F				None
A-7	0.59	May 1959	Jan.1960	1F		4F				None
A-8	1.18	May 1959	Jan.1960	2W		5F				None
A-8 Slabs 19-27	0.05	May 1959	Jan.1960		2F	15F			1W	
C	0.98	Sept.1958	Jan.1960	1W	8W 1F	2W 5F	7W			At one wide crack
D	0.91	June 1955	June 1958			4	also 3 unclassified	7	13	some at wide cracks

Remarks - W "wide" = 1/4 inch, appearance in January 1960 at 40°F

F "fine" = 1/8 inch, appearance in January 1960 at 40°F

The slabs 19-27 on type A-8 (location A-8-4) have been listed separately because it is felt that the frequent interval of cracking is the result of an abnormal factor, and not typical of this pavement in general. A plan is given on plate 6-F. Further discussion will be given in due course.

General Comparison of the Cracks

The cracks on the plain slabs tend to open and spall as time progresses, and faulting has occurred in some cases (see plate 6-G). But on the reinforced slabs, although slight spalling was noticed, the continuity of the slabs appeared to be maintained.

Thus, all cracks, fine or otherwise, in the plain slabs, must be considered as definite structural defects, whereas in the reinforced slabs this is not generally so. Because of the stiff joints, it seems very likely that the cracks in the Type C will tend to absorb the thermal movements and will therefore develop wider than those on the Type D.

Discussion of Transverse Cracks in the Type C

Many of the most badly spalled cracks are those occurring around the joints, and these do not occur in the other types of pavement. Also, they are not typical of concrete pavements in general. Perhaps, then, this pavement should be rated by the transverse cracks classified as types 1, 2 and 3. If this is so, then the amount of cracking in January, 1960, was 0.36 feet per 12 foot x 20 foot slab.

It should be noted that the majority of the transverse cracks, (types 1 and 2), amounting to 0.23 feet per slab, occurred very early in the life of the pavement. This again is not typical of concrete pavements in general because the initial rate of

cracking is most often very slow. For example, an 8 inch plain concrete test pavement in England did not start to crack until two years after construction*

Consider then, the cracks that occurred after the Type C was opened to traffic, i.e. the type 3. One accepted criterion for the evaluation of performance is the rate of cracking, because this reveals the effect of repetitions of heavy traffic loads. Plate 6-H shows the rate of cracking in Type C.

The present rate of cracking is 0.13 feet of crack per 20 foot x 12 foot slab per annum. This compares with the rate of cracking in the Type A pavements, but is worse than the performance of the Type D.

It is possible that these cracks, attributed to traffic loading, are still due to defects such as stiff doweled joints. If so, the rate of cracking does not provide a fair assessment of performance.

<u>Discussion of Transverse Cracks in the Type A Pavements</u>

Except for slabs 19 to 27, type A-8, the type A pavements are performing in a satisfactory manner, and to date the differences in slab thickness have had no apparent effect.

The widest cracks are the type 1, closest to the joints, and these could probably be eliminated by improvements in dowel setting. The occasional remaining cracks are to be expected in this type of construction and, at present, do not represent structural failure.

* J.Loe. "Performance during the First Five Years of the Experimental Concrete Road at Oxton, Nottinghamshire" Paper # 5970 Institution of Civil Engineers, 1955.

Discussion of Excessive Transverse Cracking at Location A-8-4

An interesting, although perhaps unfortunate, occurrence of uncontrolled cracking developed on pavement type A-8 at location A-8-4. A diagram is given on plate 6-F.

The first joints to crack were 23-24 and 27-28, while an uncontrolled crack occurred near joint 25-26 prior to sawing. This latter crack is similar to the type 1, in reinforced slabs, mentioned previously. The remaining cracks are typical of the types 2 and 3, and developed a few each week until the survey in August. None have occurred since. Although joints were sawn in the normal manner, many of them did not open as expected. This produced a slab length between joints of 150 feet for slabs 19-23 inclusive and 90 feet for slabs 25-27 inclusive.

It is suspected that the frequent cracking is because the slabs are behaving like short continuous sections. From data reported by Cashell and Benham*, a continuous slab of 150 feet, reinforced by deformed billet steel bars, showed an ultimate cracking interval of 50 feet, i.e. 2 cracks in 150 feet. Actually there are 10 uncontrolled cracks in slabs 19 to 23. For a continuous slab of 90 feet, from Cashell and Benham's data, no cracks would be expected. Actually there are 6 uncontrolled cracks in slabs 25 to 27. It is therefore suspected that even the "wide" joints that are operating are still stiff enough to cause cracking.

Consider the crack near joint 23-24, which is a wide joint, i.e. a comparatively free-moving joint. This crack occurred long after the concrete had gained most of its strength.

* "Experiments with Continuous Reinforcement in Concrete Pavements" H.R.B. Proceedings 1949, figure 22.

From the width of the uncontrolled crack it seems that the steel has not broken. If the steel in this uncontrolled crack were about to break, then the force per dowel in joint 23-24 nearby would be about $0.13 \times \frac{15}{12} \times 80,000 = 13,000$ pounds per dowel. (81,000 p.s.i. is the ultimate stress of the steel, obtained by Bereznicki for #4 bars, and the area of steel per foot width for this type of pavement is 0.13 square inches.)

Subgrade friction effects have been neglected in these calculations, but if the coefficient of subgrade friction is assumed to be $1\frac{1}{2}$*, then the effect of friction would amount to roughly 1,000 pounds per dowel. If this value is deducted from the force per dowel at yield point of the reinforcing steel then the resistance per dowel would be about 12,000 pounds. A force of this magnitude is equivalent to a frictional bond strength of 450 p.s.i. on a #8 dowel embedded 9 inches.*

This force of 12,000 pounds would produce a tensile stress of about 100 pounds per square inch over the cross section of the slab. Thus, the actual maximum stress produced by the dowels is probably considerably less than 100 pounds per square inch and can only amount to a fraction of the stresses due to loading and warping.

* Portland Cement Ass. "Concrete Pavement Design" page 60

* This value is in the same order of magnitude as published values. Pull-out tests carried out by Sharpe (proceedings, C.G.R.A. convention, 1957) using #4 dowels embedded 12 inches and coated with powdered graphite and oil gave a bond strength of 290 pounds per square inch. Perkins (H.R.B. Bulletin 165) obtained values of between 2,000 and 8,000 pounds for 0.1 inches removal using round steel dowels sawn from slabs three years old. The implications are that forces due to dowel misalignment at this joint are not excessive.

From these calculations, excessive cracking due to dowel resistance would not have been expected. However, excessive cracking did occur at this location and several joints remained closed. Perhaps the following explanations apply. The cracks occurred at a very early age when the concrete strength was low, but were not visible until much later. Alternatively, hidden planes of weakness due to, say, poor compaction, were present.

On the other hand, pavements might be more sensitive to direct tensile stresses from dowel restraint than would be expected from calculations considering the tensile strength of the concrete. Some authorities feel that a completely successful performance can only be obtained by using dowels of low bond resistance, accurately aligned.

Discussion of the Transverse Cracks on the Type D Pavement

The plan on plate 6-A illustrates markedly that the majority of cracks occurred before any traffic loading. In the three years after opening to traffic, only four new cracks were observed. This amounts to only 0.02 feet per 12 foot x 20 foot slab per annum.

The majority of the early cracks occurred during construction and could probably be eliminated by improvement of sawing techniques.

Judged by the rate of cracking, the performance of this type of pavement is quite satisfactory.

Discussion of Shrinkage Cracks

There were some rather spectacular shrinkage effects on this project, and examples from near station 381, pavement type C, are

given on plates 6-C and 6-D. (These slabs have since been replaced.)

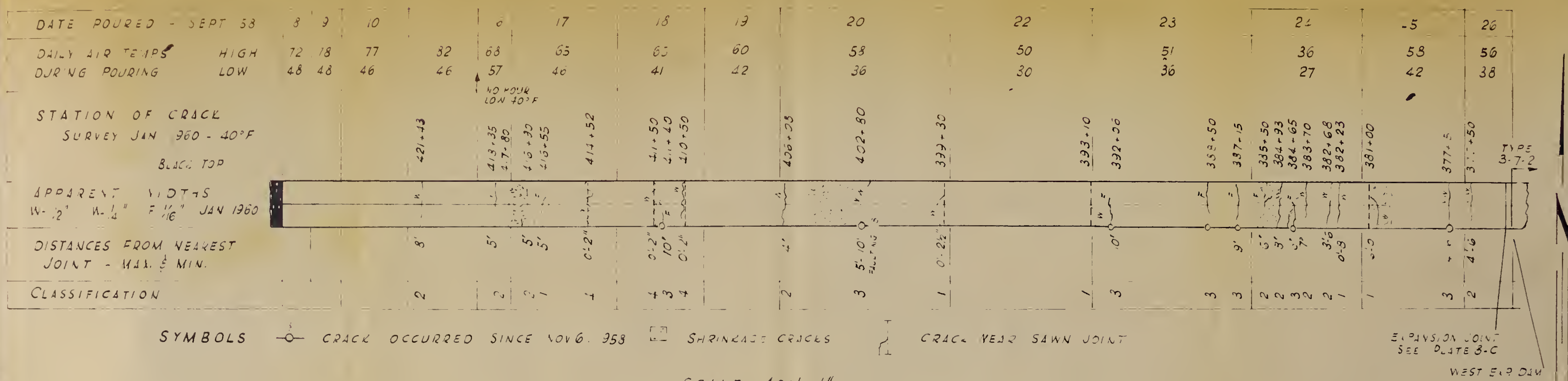
At station 381, there was a dry wind and the application of curing compound had to be delayed several hours because of mechanical finishing difficulties. This concrete was the first poured that day, and the previous night temperature had been below freezing, and therefore the aggregate was probably cold. Hence, the pouring temperature of the mix was probably low. The maximum air temperature that day was 57° and thus the temperature differential before the mix set up might have been quite high. Subsequent loss of surface moisture started the hairline cracking, and this temperature differential during set-up would tend to increase the surface width of these shrinkage cracks, when conditions of no temperature gradient were restored.

The hair-line cracking was visible the next day after pouring, but the cracks developed much worse the following spring, probably due to frost action. The flexural strength at this location was high at ten days, indicating a fast cure.

Shrinkage cracking, although not as intense as in the example above, occurred at several other locations, particularly in type C. From the plan of type C on plate 6-A it is noticeable that most of the areas of shrinkage cracks occur in the first daily concrete poured. With the concrete poured in 1959, shrinkage cracks are only noticeable at one location, slab 26, pavement type A-7. These shrinkage cracks started off as visible only a few inches in length in a random pattern, a day or two after pouring, but tended to develop as time went by.

To eliminate shrinkage, curing should be commenced immediately after brooming, at the time of the initial set.

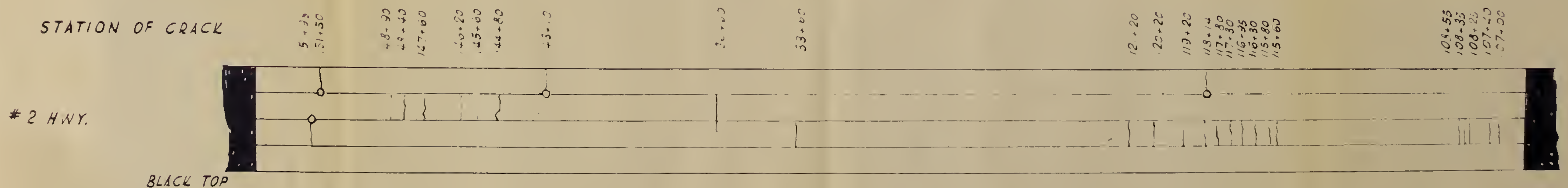
PLAN OF UNCONTROLLED CRACKS IN TYPE "C"



SCALE 400' - 1"

PLAN OF UNCONTROLLED CRACKS IN TYPE "D"

POURED JUNE 57   LAST SURVEY JUNE '58



CLASSIFICATION —o— - TYPE 3

STATION 136 + 60 TO 143 + 90 - MAINLY TYPE 7

STATION 107 + 00 TO 133 + 60 - MAINLY TYPE 6

SCALE 400' - 1"

PLATE 6-A

FIG. 1 UNCONTROLLED CRACK TYPE 3 AGE-3MO.

FIG 2. SYMPATHETIC CRACK TYPE 6

AGE 3 YEARS

PLATE 6-B

FIG. 1. UNCONTROLLED CRACK DURING SAWING OF TRANSVERSE JOINT - MORTAR FLAKING FROM AGGREGATE
AGE: 1 YEAR

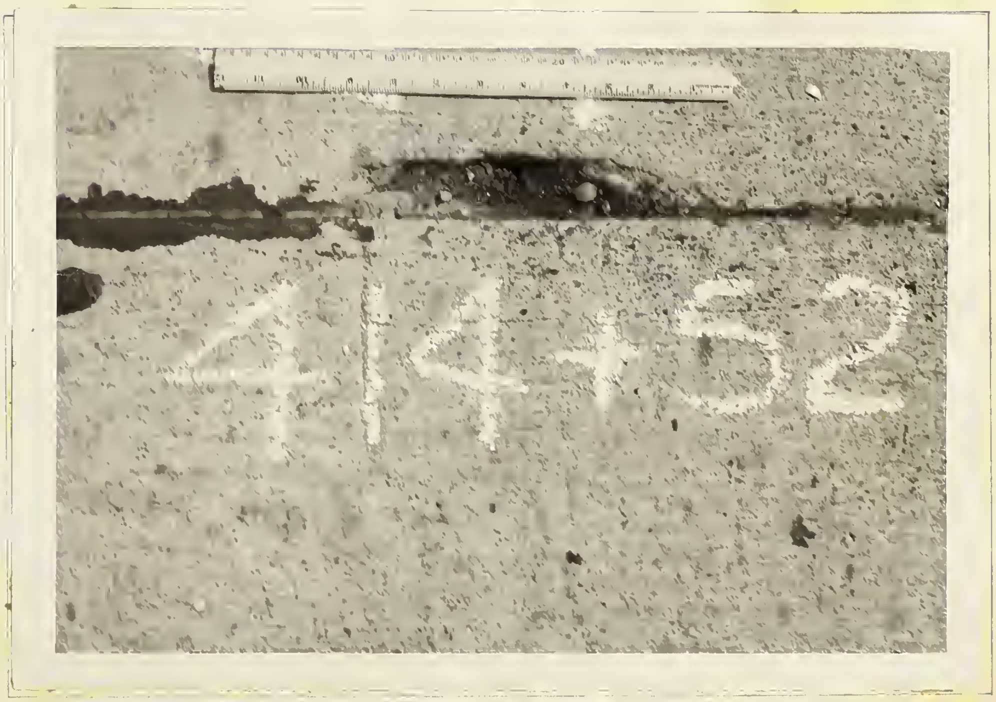


FIG. 2 LOSS OF CONCRETE BETWEEN UNCONTROLLED CRACK & JOINT AGE: 1 YEAR

PLATE 6-C

FIG 1. CLOSE UP OF TYPICAL SHRINKAGE CRACKS
AGE: 1 YEAR

FIG 2. UNCONTROLLED CRACK IN UNREINFORCED SLAB #2 HGHY
AGE: 4 YEARS.

PLATE 6-D

PLAN OF SHRINKAGE

DATE POURED: SEPTEMBER 28TH - 1958

DATE OF SURVEY - JUNE 11TH - 1959

CRACK WIDTHS AS SHOWN ON PLAN VARY IN THICKNESS FROM ABOUT 1/16" TO HAIRLINE

DEPTHS OF CRACKS: UP TO ABOUT 6" FOR WIDE CRACKS & ABOUT 2" FOR FINE CRACKS, FROM CORES

TOTAL LENGTH OF VISIBLE CRACKS: IS 145' FOR THE ONE PANEL SHOWN, i.e. 0.6' PER SQ. FT.

KEY PLAN

SCALE 1" = 40'

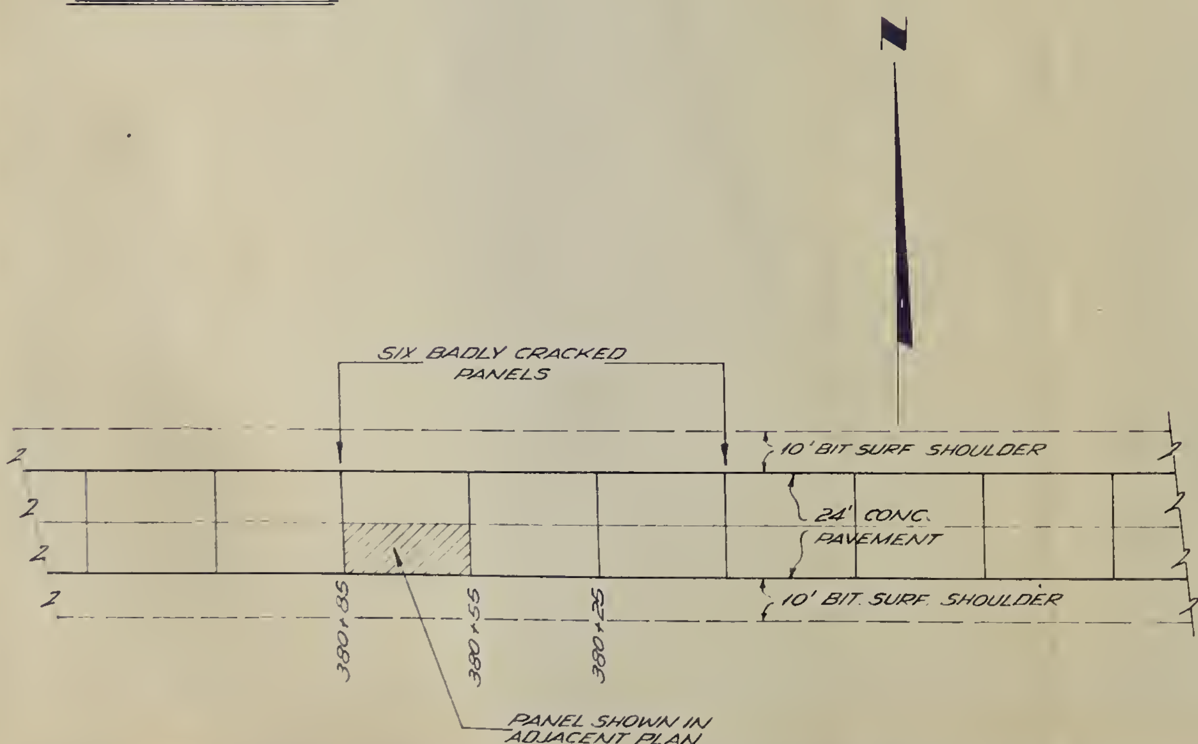
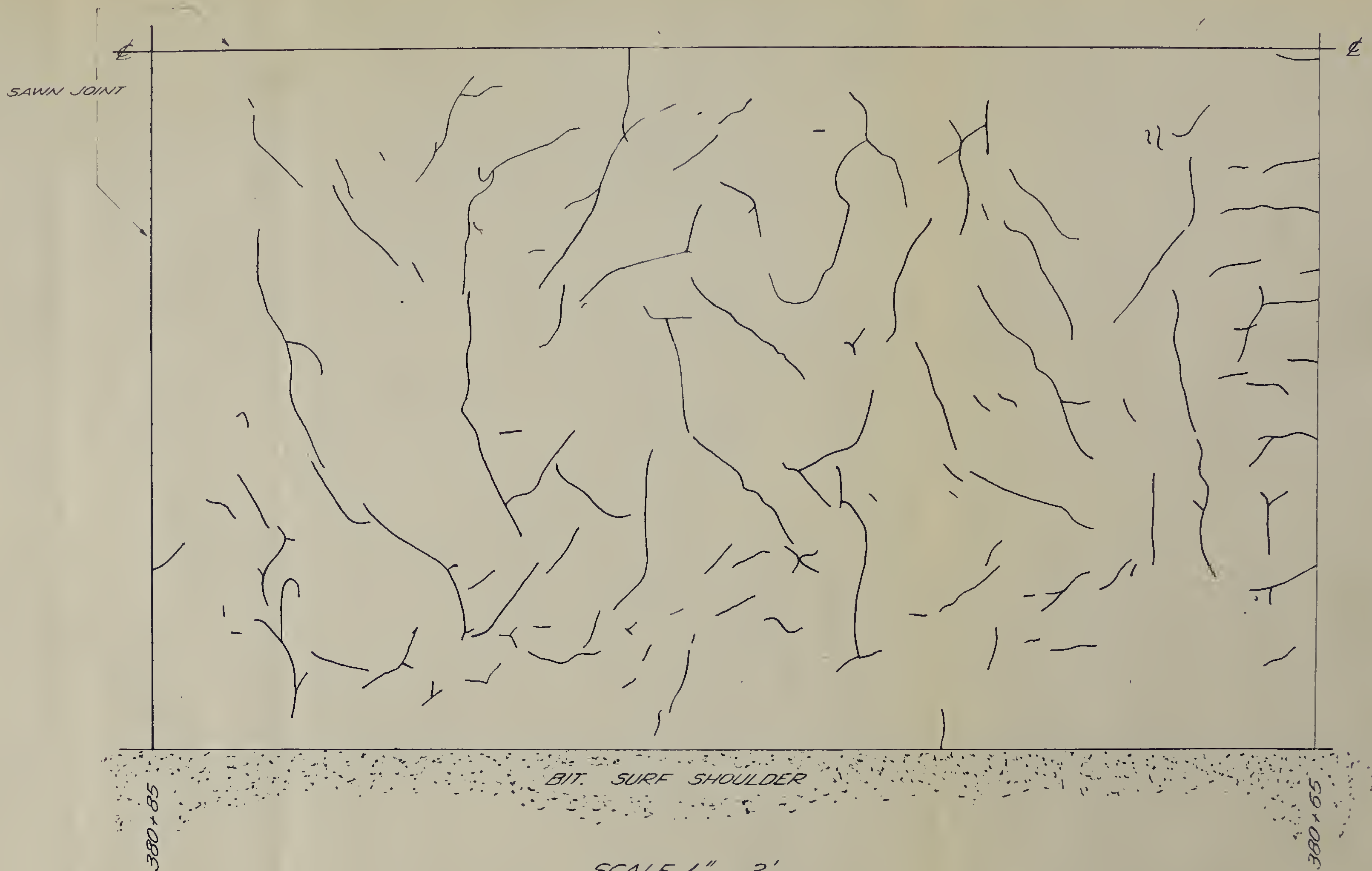


PLATE 6-E

PLAN OF SHRINKAGE CRACKS IN A 20'x 12' PANEL OF AN 8" PLAIN CONCRETE ROAD SLAB



SCALE 1" = 2'

PLATE 6-E

EXTENSIVE UNCONTROLLED CRACKING IN TYPE A.8 PAVEMENT

LOCATION A-8-4.

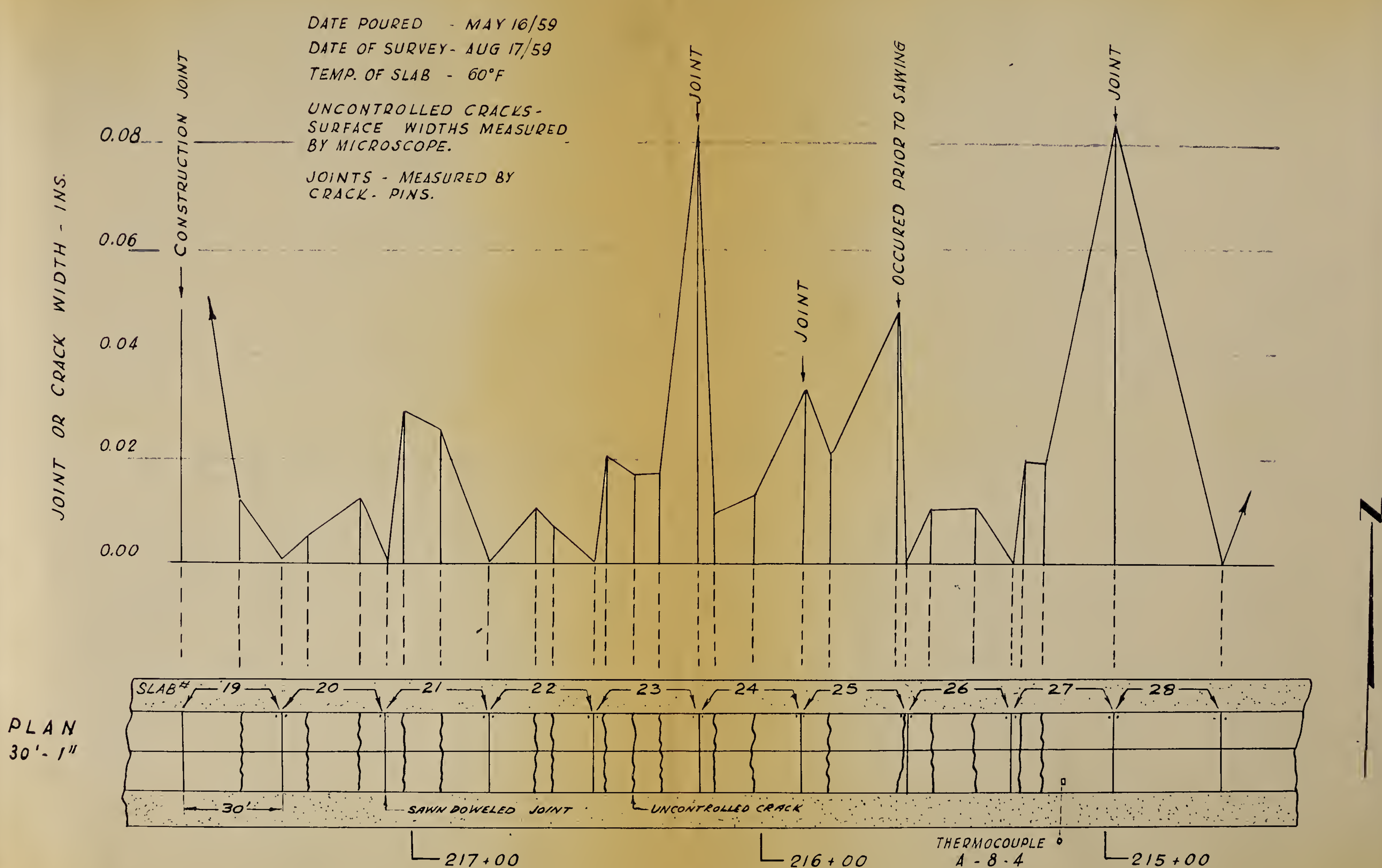


PLATE 6-F

FAULTING AT UNCONTROLLED CRACK
STATION 402+80 TYPE "C"
AGE 6 MONTHS (MAR. 1959)

PLATE 6·G

RATE OF CRACKING IN PLAIN 12' x 20' CONCRETE SLABS WITH SAWN DOWELED JOINTS

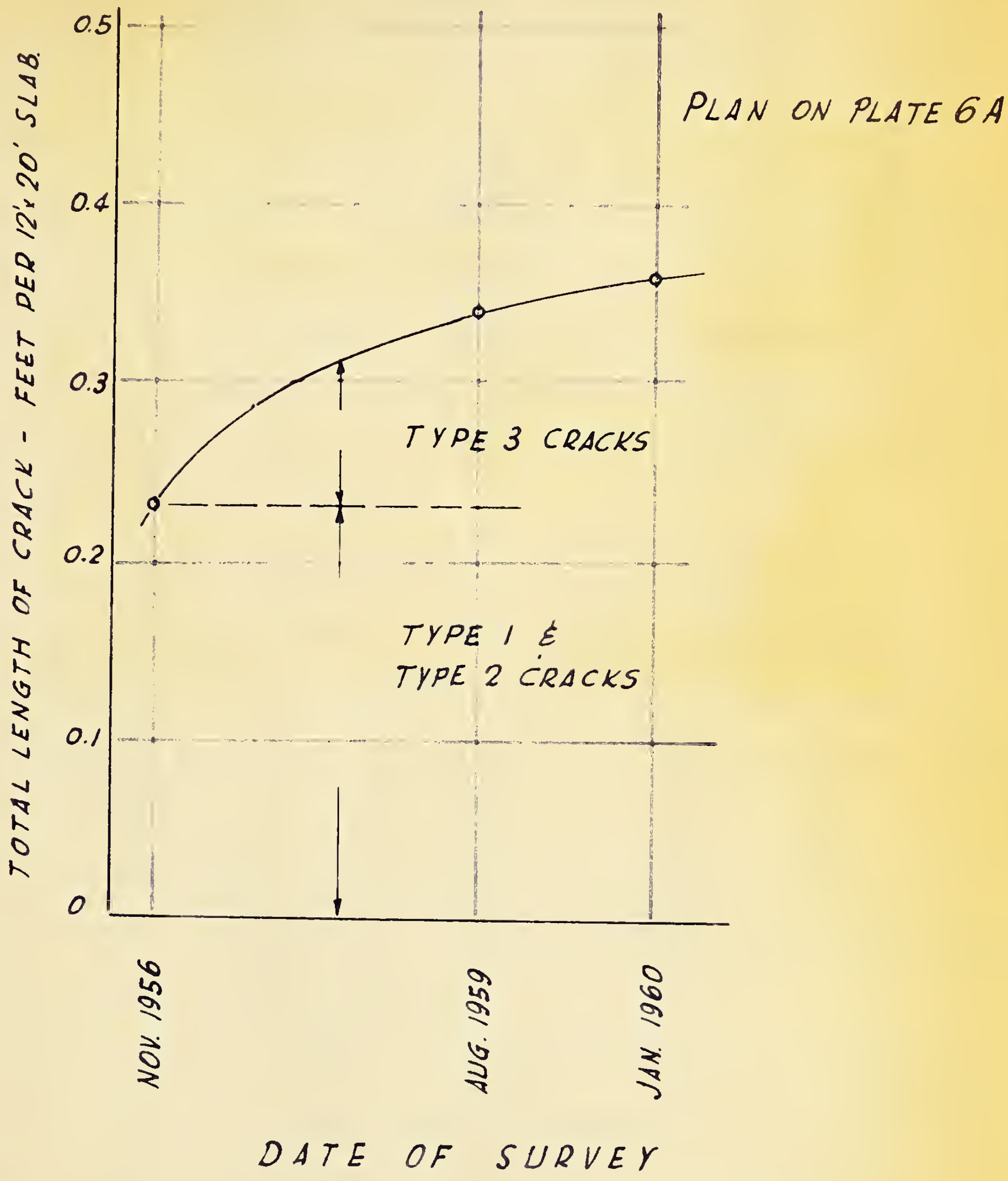


PLATE 6 - H

CHAPTER VII PAVEMENT SURFACE CONDITION

Ideally, the finished surface of a slab should present a crust of mortar which is broomed soon after the initial set and this presents a smooth riding surface with good tire grip. (See plate 3-D)

On #1 Highway, sufficient thickness of the mortar was obtained by floating, but in order to maintain the specification for surface level it was sometimes necessary for the contractor to remove high spots by hand. This tended to reduce the mortar layer which then seemed to erode rapidly, or "scale", to expose the aggregate. (See plate 7.A) Working partly set-up concrete is also likely to cause scaling.*

Scaling was very much more noticeable on the type C pavement, which represented the contractor's earliest work. In addition, the mix design used on type C did not help matters, for it tended to have less mortar than the later designs. (The mix design used was #1 on Table 3-2.)

The object in the earlier part of the project was to aim for mixes of high strength, but the later mixes seemed preferable because they finished the better.

It can be concluded that the surface finish on the type A pavements is superior to the type C. Type D is older and was finished in a different manner. Hence an exact comparison is not possible. There appeared to be no areas of excessive surface break-up on the type D pavements.

* K.Vogel ("Sawed Contraction Joints" H.R.B. Proceedings, 1953, page 136.

Surface deterioration can be detected by mechanical roughness measurements. The "HiLo Roughometer" was used by Ellert on #1 Highway in 1958. This work is described under a later heading.

FIG. I. PITTING OF SURFACE
AGE: 10 MONTHS

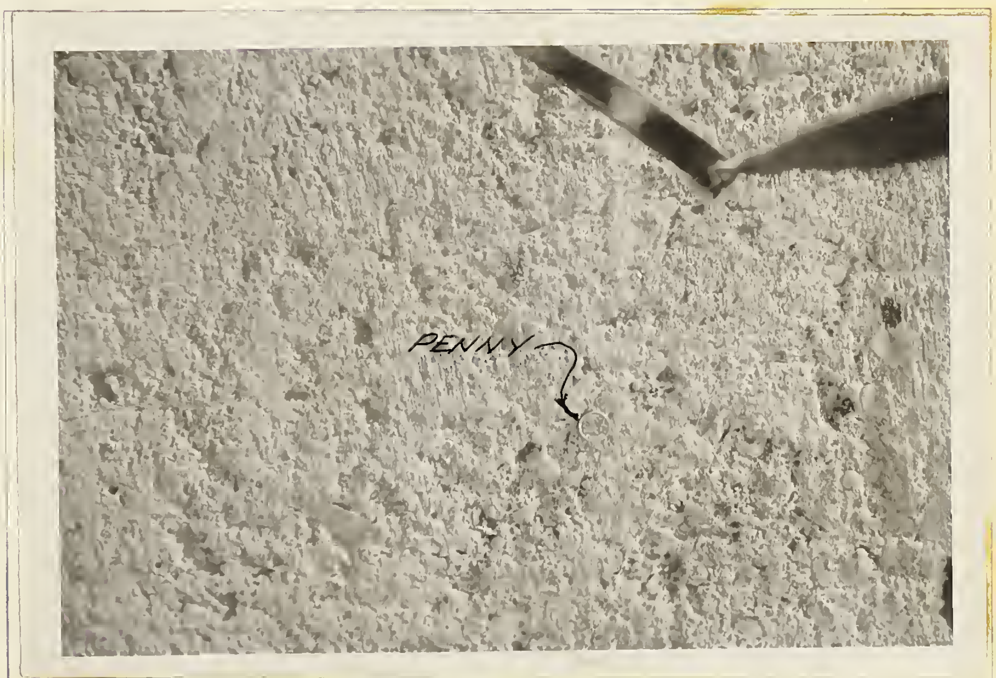


FIG. 2. CLOSE UP SHOWING EROSION OF MORTAR TO EXPOSE AGGREGATE AGE. 10 MONTHS

PLATE 7A

CHAPTER VIII VISUAL ASSESSMENT OF JOINT BEHAVIOUR

General Rating of Transverse Joints

A survey of joints in August, 1959, revealed that the majority of transverse joints in types A and C pavements were in "good" condition. An example is given on plate 8-A, figure 2.

The joints in type D pavement were in poorer condition than those in the types A and C and a driver was more conscious of them when driving in a car. Their general condition can be rated as "satisfactory" as compared with the general rating on types A and C of "good". Examples are shown in plate 8-B-1.

Uneveness and Faulting

No joint faulting was observed on types A and C but there was slight faulting at some joints in the type D (see plate 8-B-2, fig.2).

The formed joints in type D were more uneven than the sawn joints (see plate 8-B-2, figure 1) and the same applied to the daily construction joints in the types A and C. These uneven joints in types A and C were planed by the Contractor in June, 1959. A photograph of a planed construction joint is given on plate 8-A, fig.1.

Joint Sealer

Uneveness at the joints is also the result of excessive joint sealer. On plate 8-C, figure 1, an excess of sealer was rapidly spread by the traffic until it became a strip 7 inches wide and started to collect small stones. The tendency of the joint sealer to flow is revealed by plate 8-C, figure 2, which shows an expansion joint on a five per cent grade in hot weather.

Pumping

Observations were made after heavy rain in August on the type D pavement, and no pumping was noticed. However, the wider joints looked as if they had been re-sealed shortly before the survey and it is suggested that another pumping survey should be

carried out during the spring break-up, 1960. No soil stains could be seen during dry periods of weather.

On #1 Highway observations were made several times during summer 1959 at many different locations and no pumping was noticed.

Joint between the Shoulder and Slab on #1 Highway

Near the end of August, after a day or two of rain, oozing water was seen from the joint between the shoulder and the concrete slab at twenty-three locations on all the types of construction on #1 Highway.

The worst of these locations were on pavement Type A-7, station 226 + 10, south side, and on pavement type A-8 at station 221 + 70, north side. Between August 23rd and September 19th, 1959, when observations discontinued, oozing water was noticed on twelve days from station 221 + 70, and eight days from station 226 + 00. Photographs are shown on plate 8-D.

At the remaining locations, water was seen to ooze on two days during this twenty-eight day period.

It should be noticed that the two worst cases occurred near the bottom of a five per cent grade, about 2,000 feet long. It is felt that water enters through the joints into the base and runs out at some point of weakness at a lower level, perhaps aided by a surface heat from sunshine.

The greatest quantity of water was probably entering via the shoulder joint itself since the sealing on the transverse joints was new and the weather not cold enough for them to open wide. Close to the edge of the slab, the shoulder felt

spongy underfoot, and in the locations shown in the photographs water could be squirted out by stamping with the heel.

On type B near station 374, gradient and length of gradient are about the same as near station 226, yet oozing occurred only on two days. The base material in both cases came from the same pit, the MacIntosh pit, and according to the Engineers from the Department of Highways there is no significant difference between the base materials near station 374 and 226. It is possible that the expansion dam on the grade above station 374 intercepted water running downhill through the base. However, a base that cannot freely remove infiltrated water is not performing in an entirely satisfactory manner. Only time can tell if the specification already in use is satisfactory for concrete bases.

Another bad feature is the tendency of the shoulder to settle away from the slab. The photograph on plate 8-E, figure 2, was taken only three months after construction and shows a settlement of half an inch.

It is possible that these problems of water infiltration and settlement could be avoided by designing a joint with the bituminous surfacing lapping onto the concrete slab, and by strengthening the shoulder immediately adjacent to the slab.

<u>Centre Line Joint Behaviour</u>

On #1 Highway, crack pins* were installed in pavement types A-7 and A-8 to examine the cracking behaviour of the centre line joint. From the readings obtained it was concluded that at none of the locations had the centre line joint cracked and widened.

* See Chapter X

However, the Berry Gauge used was probably too insensitive to detect a crack of hairline width.

Hence, three cores were taken on the centre line to discover if any centre line joint cracking had actually occurred. None of them showed any centre line cracking either on thpe B or type C. The centre line coring on type B was at station 351 + 59, and on type C at stations 380 + 50 and 417 + 50.

At one location, however, it seemed certain that the centre line joint was cracked because of the oozing water (see plate 8-F.) At other locations on type C, transverse cracks occurred on half roadway width only, and hence it is likely that the centre line joint cracked here also.

In general, it seems that the centre line joint does not readily crack, but since no longitudinal cracks have occurred (except one on type B) this is no cause for concern.

On #2 Highway the longitudinal joints were formed with no tie bars, and it appears that they are slowly increasing in width.

Elutriation of the Fine Base Material through the Transverse and Centre Line Joints

The photographs on plate 8-F were taken near station 221 on type A-7, previously mentioned in connection with water oozing from the shoulder joints.

The water from these joints is not being pumped out even when heavy trucks pass but, however, fine material is being elutriated (note the soil stains plate 8-E, figure 1).

Only time will tell the seriousness of this effect.

JOINTS IN TYPES A AND C PAVEMENTS

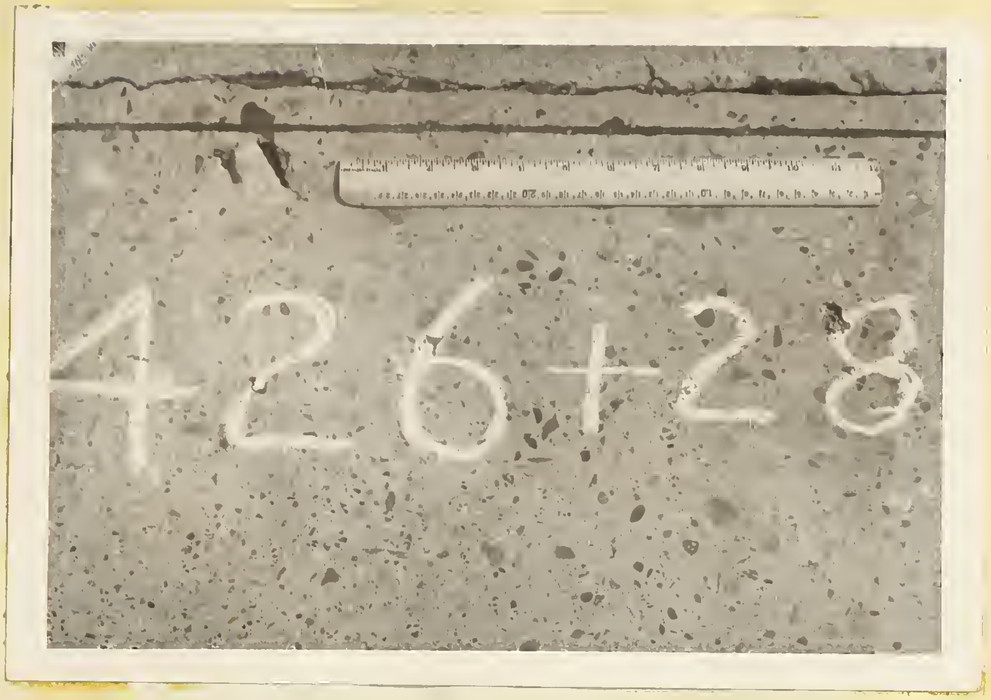


FIG 1 [illegible] INED CONSTRUCTION JOINT
(SAWN DUE TO A MISUNDERSTANDING)

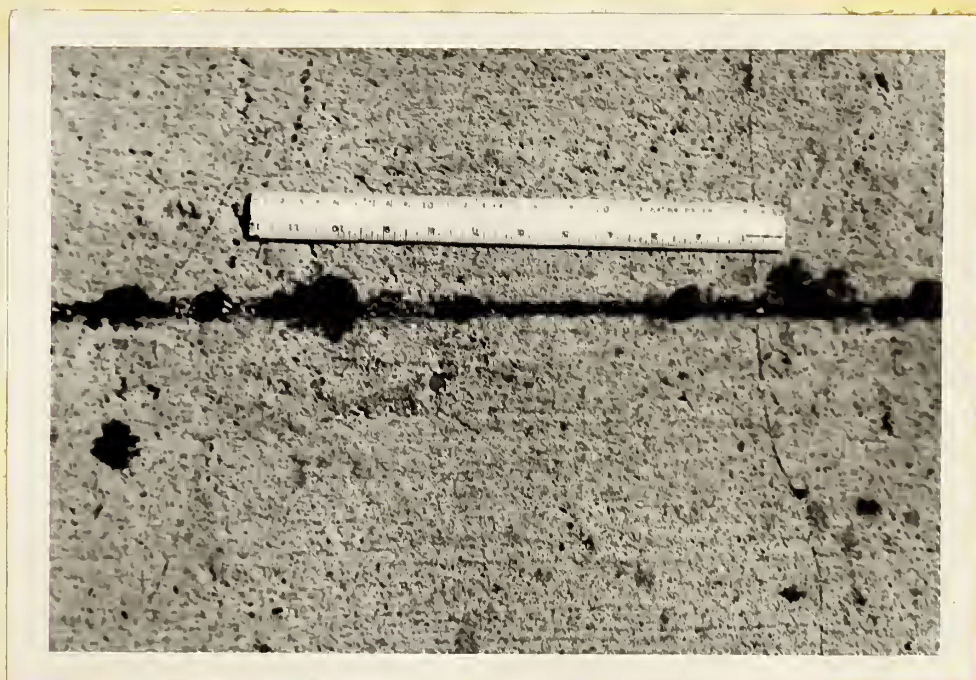


FIG 2 TRANSVERSE CONTRACTION JOINT IN GOOD CONDITION
[illegible] YEAR

PLATE 8-A

TYPICAL JOINTS IN TYPE "D"

FIG. I TRANSVERSE JOINT AGE - 2½ YRS.
NOVEMBER 1957

FIG. 2 TRANSVERSE AND LONGITUDINAL JOINT AGE - 3 YRS.
JULY 1958

PLATE 8-B,

JOINTS IN TYPE "D"

FIG. 1 FORMED JOINT AGE 3 YEARS

FIG 2.
SLIGHT FAULTING
AGE. 3 YEARS

PLATE 8. B2

JOINT SEALER

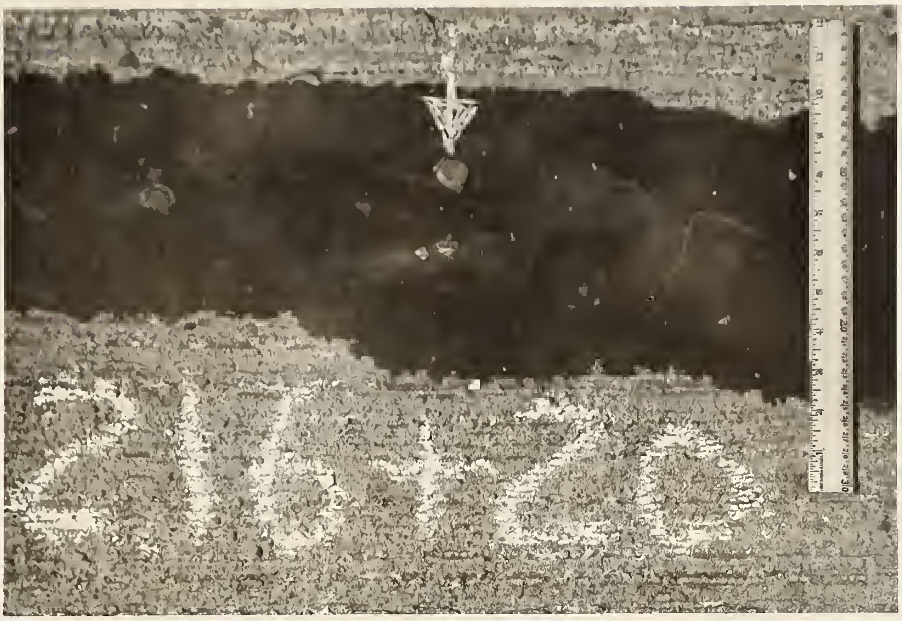


FIG. 1. SURPLUS CONTRACTION JOINT SEALER SPREAD BY TRAFFIC.

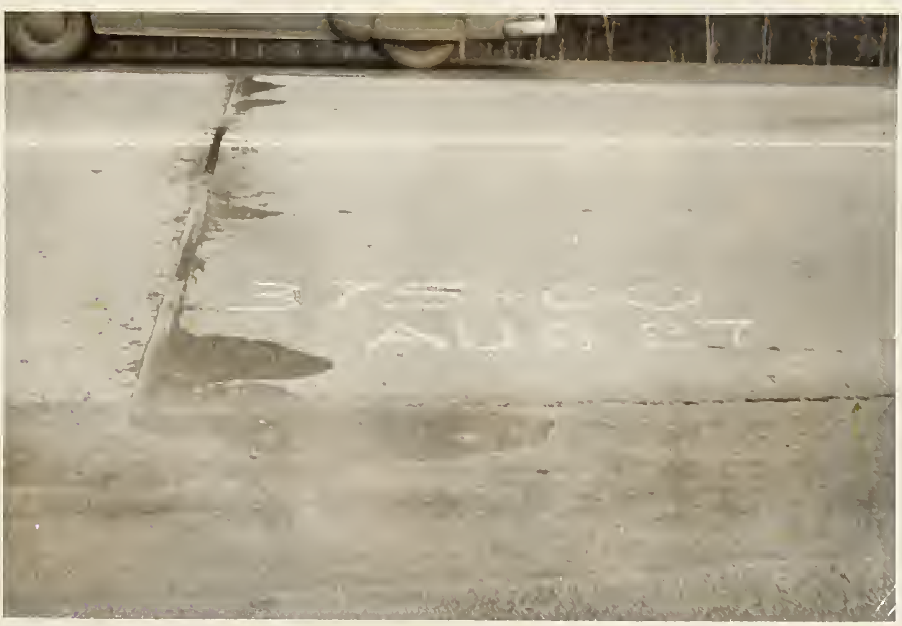


FIG. 2 FLOW OF JOINT SEALER FROM EXPANSION JOINT ON 5% GRADE (LOCATION SHOWN ON PLATE 6-A, TYPE C)

PLATE 8-C

WATER OOZING FROM SHOULDER JOINT

FIG 1 WATER OOZING FROM SHOULDER JOINT
TYPE A-7 AGE 3½ MONTHS.

FIG 2. WATER OOZING FROM SHOULDER JOINT
TYPE A-8 AGE: 3½ MONTHS

PLATE 8-D

SOIL STAINS AT JOINT AND SETTLEMENT OF SHOULDER

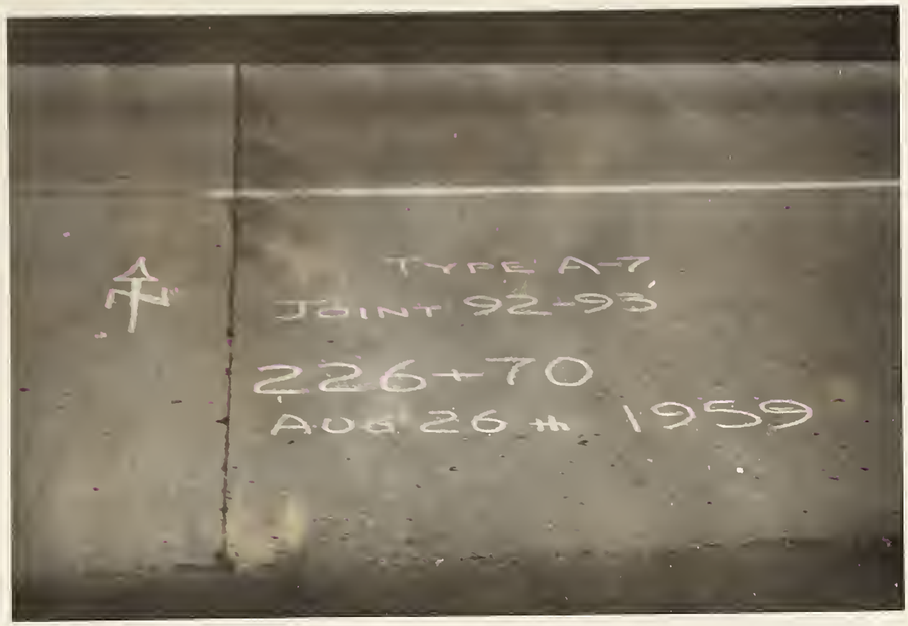


FIG 1 SOIL STAINS AT TRANSVERSE JOINT

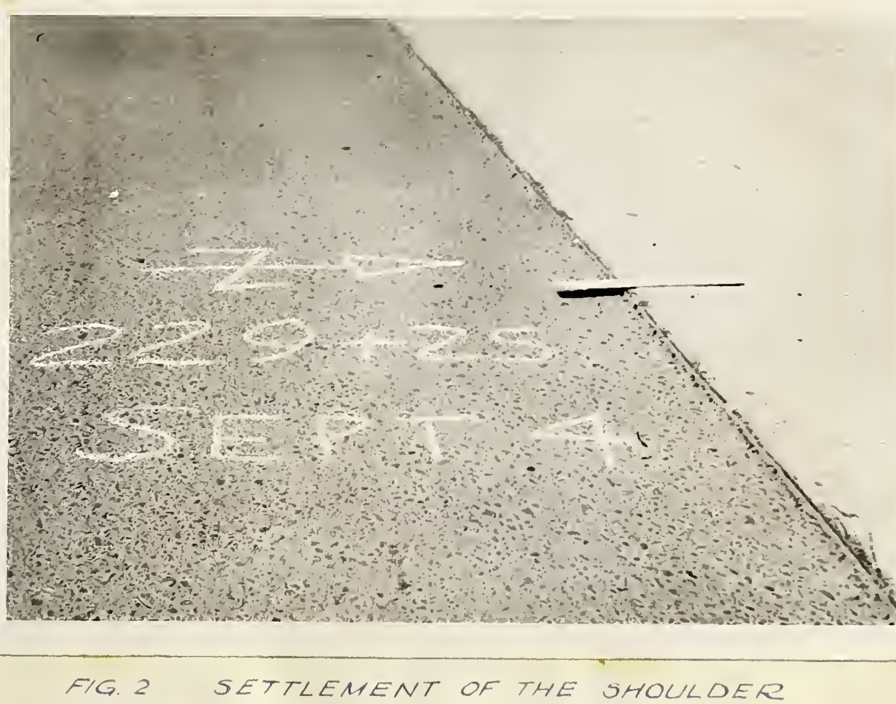


FIG. 2 SETTLEMENT OF THE SHOULDER
TYPE A-7 AGE: 3 MONTHS

PLATE 8-E

WATER OOZING FROM JOINTS

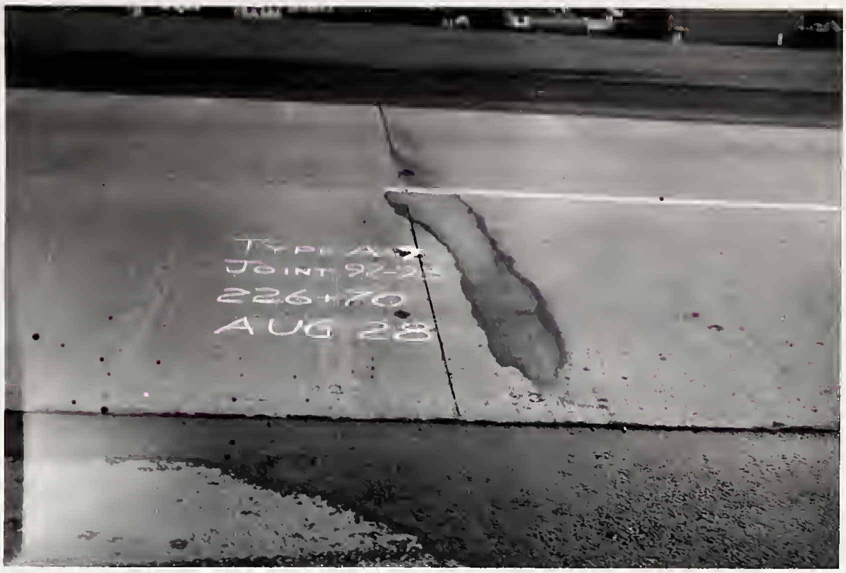


FIG 1 WATER OOZING FROM TRANSVERSE JOINT
TYPE A-7 AGE: 3½ MONTHS

FIG 2 WATER OOZING FROM CENTRE LINE JOINT
TYPE A-7 AGE: 3½ MONTHS

PLATE 8-F

CHAPTER IX RIDING QUALITY MEASUREMENTS

The test roads were located adjacent to asphaltic concrete pavements constructed at about the same time, and this should enable a good comparison of the riding quality and the deterioration in riding quality for these types of construction to be made.

Methods of Measuring Riding Quality

Assessment of riding quality has been obtained mainly by two different methods.

In Britain and some parts of the U.S.A. the Profilometer has been used. By means of articulated sets of tandem wheels spanning usually about ten feet, a fixed point on a beam is established at a set distance above the average road surface. A small recording wheel acts on the road surface below this fixed point, and an automatic device produces a continuous surface profile with reference to the average road surface. Sometimes a classifier is included, which counts the number of irregularities greater than 0.1 inches, 0.2 inches, etc., and in addition integrates the profile, producing an "Irregularity Index" "q" in inches/mile. A profilometer q value for a "good" road would be less than 75 inches per mile and is said to correspond approximately to a specification requiring no irregularity greater than 3/16 inch in ten feet.*

An alternative method, developed by the Bureau of Public Roads, is said to simulate more closely the effect of a pneumatic tire on a travelling vehicle and consists of a single wheel built into a heavy trailer. The vertical oscillations of the single

* Road Research Laboratory "Concrete Roads" paragraph 18.5

wheel within the heavy frame are recorded autographically and an irregularity index in inches per mile is produced. For a good road, it would be below 75 inches per mile.* Although the Profilometer travels very slowly, the Trailing Wheel can be towed at normal driving speeds.

A fairly full description of these more commonly used methods has been mentioned because the HiLo Roughometer was a newer and much simpler device, and it is important to compare its action to the methods above.

Description of HiLo Roughometer

This was a perambulator, with two wheels on a span of sixteen feet in a forward-going direction. At mid span there was a vertical rod carrying a small recording wheel riding on the road surface. The vertical movement of this rod actuated a pointer over a segment of a dial. This dial was divided by seven lines. The pointer at centre, zero, showed the recording wheel concurrent with the points of pavement contact of the span wheels. The first line each side showed the recording wheel to be 1/16 inch, the second line 1/8 inch, and the third line 1/4 inch, either higher or lower than the straight line joining the span wheels.

The perambulator was pushed at slow walking speed, and each time the pointer came from zero into the space between the 1/16 inch and 1/8 inch lines on the dial and reduced again to zero, a roughness number of one (R_1) was recorded. Similarly, a roughness number of two (R_2) was given to the space between

* F.Holloway "Road Roughness Measurements on Indiana Pavements" Proceedings, Purdue Road School 1956

the 1/8 inch and 1/4 inch lines, and a roughness number of four (R_4) outside the 1/4 inch line.

The roughness numbers were totalled for each 500 feet and this total used for subsequent calculations (called R500).

Ellert's Analysis

The readings using this roughometer were taken by Ellert in 1958, who analysed them in the following manner. He stated that R_1 represented a projection of 5/64 inch, R_2 a projection of 10/64 inch and R_4 a projection of 20/64 inch, i.e. the roughness number is proportional to the height of the projection over which the recording wheel has travelled.

Each projection will act upon both span wheels, and in each case will result in a low reading proportional to half the height of the projection. Thus, the sum of the height of the projections is proportional to half the sum of the R.values. Actually, "q" = $\frac{R500}{2} \times \frac{5280}{500} \times \frac{5}{64}$, or q = 0.40 x R500 inches/mile.

Ellert obtained a set of readings along the wheel path of each lane, and the value presented is the average for four lanes. His results are plotted on plate 9-A.

Discussion

Actually all three instruments mentioned above are "profileometers" in the sense that they measure undulations relative to the average surface covered by their span. Heavy damping in the Trailing Wheel places it into this category. Thus they are most accurate for measuring small, isolated, undulations, and will give distorted values for small changes in grade over distances greater than the span length. The irregularity index therefore depends to some extent upon the span length. The HiLo Roughometer is probably as good as any other method for detecting deterioration of riding quality,

and for comparing one type of construction with another; providing reckoning is included for areas of settlement. However, before proceeding with extensive Roughometer measurements, a check on reproductibility of results should be made.

The data shows a gradual improvement from the time when the contractor commenced on type C until he finished that year on type A-6.

The survey was carried out soon after the completion of construction, and before any major traffic wear, or joint deterioration. Thus, if the joints themselves had been a factor, type B could be expected to run smoother than the type A-6, which is not the case. It can therefore be concluded that the major source of uneveness occurred during construction.

Factors occurring during construction that have been found to have a significant effect upon riding quality are:*

1. Settlement of the forms
2. The method of dumping the concrete between the forms. For example, if the concrete is dumped systematically in one spot, uneven compaction occurs, which the finishing cannot completely eliminate. When heaps are dumped too close together honeycombing occurs.
3. Equipment improperly operated or out of adjustment.

Effects such as this must therefore constitute the majority of roughness measured by Ellert. The writer did not observe the construction in 1958 when the worst riding concrete was poured, but in 1959 the method of setting the forms had not changed, and there was nothing obviously at fault with the operation of the

* R.Kirkham "Factors affecting the Riding Quality of Machine Laid Concrete Roads". Road Paper 41, Proceedings Institution of Civil Engineers, 1953.

paver mixer and spreader. Hence, most of the difficulties in 1958 on the type C pavements must have been caused within the operation of finishing. As previously pointed out, the mix used on type C did not finish so well as the mixes used later in the project.

Factors that cause rough riding as time goes by are mainly the result of joint deterioration, uncontrolled cracking, and settlement. The joints and uncontrolled cracks can fault, spall, and the sealer can pick up small fragments. In the final stages, the pavement might consist of a collection of slab elements slightly tilted in different planes, with uneven waves of much larger amplitude due to deep seated settlement.

Except for the settlement, there seems to be no reason why the Roughometer should not give some measure of the progression of these factors. Another variable not mentioned above, is roughness caused by slab temperature warping when short slabs are used. This can be eliminated by running the survey when there is no temperature gradient.

Conclusions

If a check shows that results can be reproduced, the HiLo Roughometer is suitable for assessing riding quality. The value obtained is not directly comparable either with the Profilometer or the Trailing Wheel and perhaps a correlation could be arranged in the future. Pavement type A-6 can be considered to have good riding quality and it gave an average Roughometer value of about seventeen inches per mile. Good riding quality would be equivalent to seventy-five inches per mile with the Trailing Wheel and Profilometer. Hence as a first estimate, a value of

seventeen inches per mile with the Roughometer can be considered equivalent to about seventy-five inches per mile with the other two methods.

The results obtained from the survey, which was taken soon after construction, seem to indicate that the majority of unevenness was not due to the joints or cracks, but resulted from difficulties in finishing the concrete. This was probably partly due to the inexperience of the labour, and partly to the harshness of the concrete mix design.

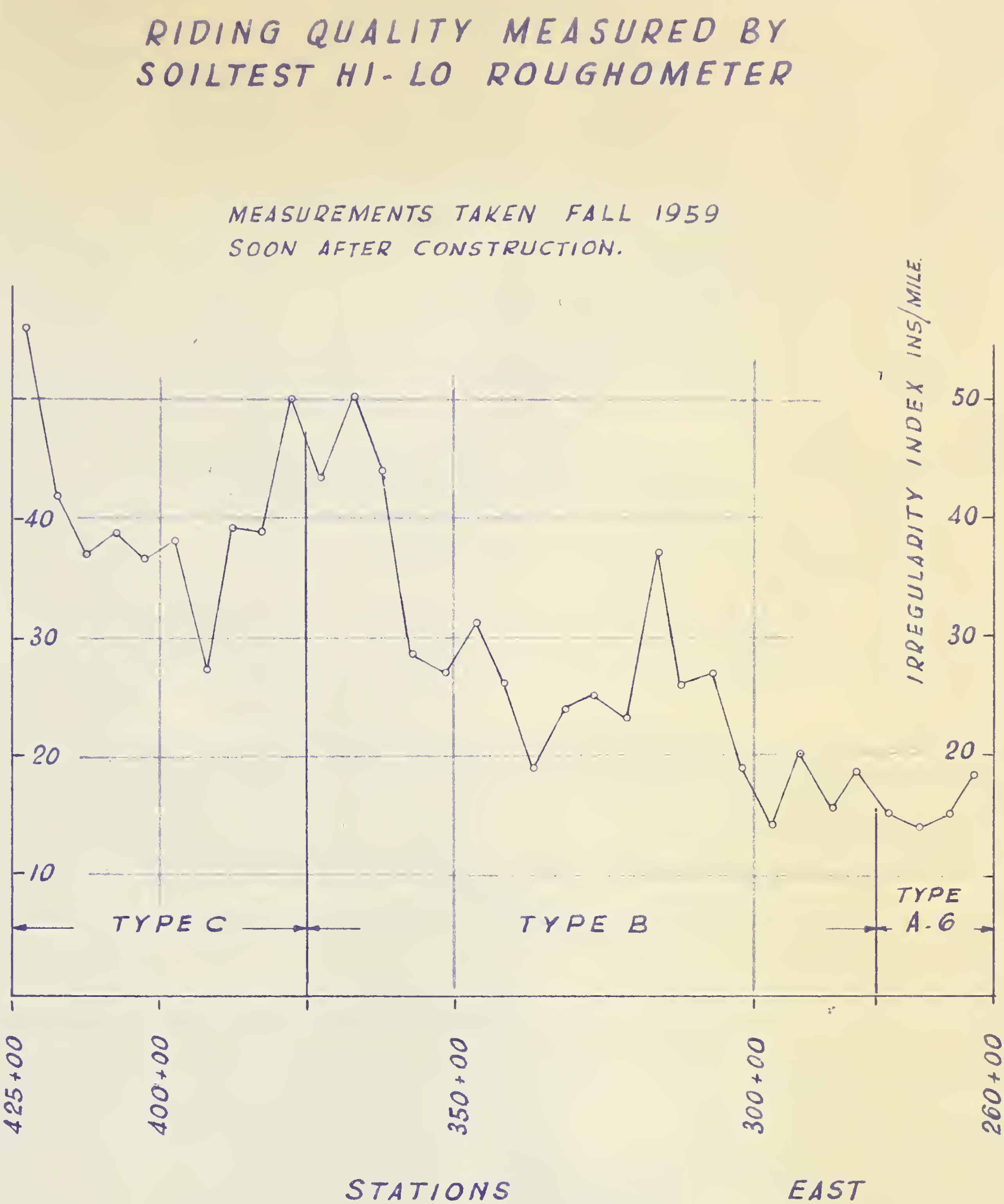

RIDING QUALITY MEASURED BY
SOILTEST HI-LO ROUGHOMETER
MEASUREMENTS TAKEN FALL 1959
SOON AFTER CONSTRUCTION.
IRREGULARITY INDEX INS/MILE.
50
40
30
20
10
TYPE C
TYPE B
TYPE A-6
425+00
400+00
350+00
300+00
260+00
STATIONS
EAST

PLATE 9-A

CHAPTER X JOINT MEASUREMENTS

Preamble

The behaviour of the pavements can be satisfactorily evaluated by means of observations over a period of years correlated with measurements of riding quality. However, many hundreds of joint measurements have been taken on these Alberta test roads and these measurements are to obtain better understanding of the behaviour of short reinforced slabs, utilizing the contraction joint design.

It has been discovered from joint condition surveys in the U.S.A. that some joints and sealers fail before others, and that joints remaining in good condition show the least movement.* A very useful study would therefore be a correlation over several years between joint width variations and joint performance assessed by visual studies.

A detailed analysis of joint behaviour was started by Ellert, and his ideas have been extended by the author. The locations examined are shown on plate 3-A. Each location consisted of a string of consecutive joints, instrumented so that widths could be measured.

Method of Joint and Crack Width Measurements

Measurements were made by means of a direct reading mechanical type of strain gauge on an eight inch gauge length. (See plate 10-A) The points of the strain gauge fitted into holes either drilled or punched on the surfaces of crack pins or Ramset discs, which were installed as shown in plate 10-B.

The crack width or joint width was taken as: "the difference between the present gauge reading and a reading taken at zero

* E.T.Perkins "Test Projects Constructed Utilizing the Contraction Joint Design" H.R.B. Bulletin 165, page 41

width", i.e.

crack width, w, = present reading - initial reading

A crack pin was inserted into the freshly poured concrete on each side of the joint to be examined. Thus for the crack pins the initial reading could be obtained before the joint had cracked. The Ramset discs were held by a Ramset pin, shot in after the concrete had hardened and cracked. Hence, with the Ramset discs, the initial reading had to be estimated. At high temperatures the gauge readings were almost independent of temperature, and thus the crack was assumed to have zero width.

Joint Numbering

Prior to 1958, each joint at each location was given a number. For example, location C-14, joint #2, can be readily located when the plan of C-14 is available. (See plate 10-G.) In 1959 each individual slab was given a number and the joint was picked out by the number each side.

For example, Type A-8 joint 182 - 183, South, would mean the South end of the joint between slabs 182 and 183, in pavement Type A-8.

Slab Temperature Measurements

On #1 Highway, slab temperatures were measured by means of copper constantan thermocouples, installed prior to pouring. A thermocouple unit consisted of one hot junction one inch from the surface of the slab, another one inch from the slab bottom, and a third in the middle of the slab. These wires were tied to a stake driven into the base prior to pouring. The end of these wires connected to an Amphenol connector located in the

side slope as shown on plate 10-C.

The temperature measuring equipment consisted of a null reading potentiometer, a six way switch, and a thermos bottle, and was carried in a box in a car. The thermos bottle, containing ice and water, was used for the cold junction. A fuller description of this method of measurement has been given by Berznicki.

On #2 Highway, slab temperatures were measured by inserting a thermometer $1\frac{1}{2}$ inches down into a close-fitting cardboard tube fixed into a wide joint.

Air temperatures were measured by a mercury in glass thermometer, shielded by the body from the sun.

<u>Joint Width Variations on Pavement Type A-8</u>

The most intensive study of joint width variations was at location A-8-6, where seventeen consecutive joints were examined by means of crack pins. A plan and details of the location, which is at zero grade, are presented in plate 10-D. Data was recorded to investigate the following behaviour:-

1. Joint width variations during curing
2. Joint width variations due to temperature changes
3. Joint width variations due to weather changes
4. Joint width variations between the north and south sides

Additional data is presented from other measuring locations to correlate with the data from location A-8-6.

<u>Joint Width Variations during Curing</u>

At location A-8-6, initial readings were taken when the concrete was one day old before removal of the forms. During the night of this day cracking started, and at an age of two days the construction joints and joints 182 - 183 and 187 - 188 were

visible as cracks.

Joint width measurements were taken at fairly close intervals of time for the first few weeks, and three sets of readings at one particular temperature were selected to illustrate the shrinkage behaviour. The choice of readings at a single temperature enabled a comparison of shrinkage effects alone to be made. This data is presented on plate 10-D.

The diagram shows that shrinkage was not complete at two days, although most of the joints had cracked. As mentioned previously, only two cracks were clearly visible.

After eight days, the cracking pattern was well developed, and the pavement joints had formed cracks of two distinct sizes. These can be termed "wide cracks" and "narrow cracks" and although they were not measured, the construction joints appeared to be wide cracks.

The pavement had thus arranged itself into "units", either two, three, four or five slab panels in length. The units were separated one from another by the wide joint cracks, and each unit contained several narrow joint cracks.

After twenty-five days conditions in the slab had changed very little, except that the five slab units had broken down into smaller units. One five-slab unit originally included slabs 183 to 187, and the other slabs 171 to 195.

Shrinkage must have been complete at eight days because the average crack width at eight days and twenty-five days was the same. Therefore the five slab units were most likely broken down by frictional forces.

Joint Width Variations Due to Temperature Changes

The results for location A-8-6, presented on plate 10-E, figure 1, were taken after shrinkage had been adjudged complete.

The graphs show that at temperatures above the pouring temperature the cracks are very small. Also apparent is that width variations in the wide cracks are greater than in the narrow cracks.

Joint Width Variations due to Weather Changes

Study of the previously mentioned set of slabs was continued during the hot weather in July, and into fall.

Readings were taken in the cooler early morning during this hot summer period, and are compared with readings at the same temperature in June and September. The results are presented on plate 10-F, figure 1. Although all readings were taken at the same temperature it can be seen that the cracks in July are generally much wider than in June or September. This means that the slabs must have shortened during the hot summer weather.

The average daily temperature in July was higher than the pouring temperature. Hence, during most of the daily period, the cracks were completely closed up. This resulted in a compressive stress within the slabs, causing them to shorten.

The results show some recovery of this shortening during the cooler weather in September when there was no temperature induced compressive stress within the slabs. This phenomenon of compression under sustained load and partial recovery when load is removed has been described by the Bureau of Reclamation*. The deformation results from elastic strain which is immediate and fully recoverable, and creep strain. Creep depends upon the length of time during which the load is applied and is only partially recoverable.

This seasonal slab shortening and recovery from summer to fall,

* "Concrete Manual" Chapter I

could be called "weather creep".

Loss and gain of moisture could also have been a factor contributing to the effect illustrated by these graphs, but there are reasons for believing it was slight. This factor is, however, more conveniently discussed under "Average Joint Width Variations".

At the end of the summer, the system of units still existed, but the effect of the weather changes had been to widen many of the narrow cracks and to reduce the wide cracks. Plate 10-F, figure 2, perhaps illustrates the effect more vividly and shows that the behaviour of the joints in the fall is considerably less orderly than in the spring. Data on plate 10-G, which is for the unreinforced slabs, Type C, provides a comparison from the fall of one year to the fall of the next, and also illustrates the effects mentioned above.

Differences in Joint Widths Between North and South Sides

Typical results which are compared on plate 10-E, figure 2, show that at location A-8-6, cracks on the north side are in general slightly wider than those on the south side. Due to the horizontally curved alignment, each slab is about four inches longer on its north side than on its south side. This small difference alone cannot explain the differences in crack width.

Seeking a cause for this effect suggests the following considerations:

1. Any possible difference in heat from the sun.
2. " " " " the concrete mix.
3. " " " " curing technique.
4. Effect of geometry of curve.

Since the crossfall was uniform the first cause is unlikely. The concrete was laid in one slab, both north and south lanes at the same time, and the curing compound appeared to be evenly applied. Thus reasons two and three also seem unlikely.

Other investigators have attributed wider cracks at the inside of a curve as cooling takes place due to the curve trying to "straighten out". However, this cannot explain the behaviour on #1 Highway because the cracks widened on the outside of the curve as cooling continued. This was proved by comparing a plot of the average joint widths on north and south sides, at different temperatures (see plate 10-H).

Thus, as present, there seems to be no logical explanation.

Unfortunately, at no other location was crack width data on both sides obtained sufficient for comparison.

Discussion of Results

The most significant observation is the preference of the road slabs to form during shrinkage a well defined system of wide and narrow joints. The optimum spacing between wide joints is about 90 feet, which corresponds to the spacing of formed "control joints" used by some State Highway Departments.*

Data provided by Ellert and given on plate 10-J supports the data already presented.

As an explanation for this behaviour, consider the action of the subgrade friction during shrinkage. The construction joints

* E.Coppage, Jr. "Sawed Joints in Concrete Pavements: Progress and Problems" H.R.B. Bulletin 132, page 31, 1956.

at each end of the slab can be considered as the first "free ends" of a slab. The slab therefore tends to shrink away from the construction joints and the subgrade friction is directed in opposition to this shrinkage. At a certain distance from the construction joints, the force of subgrade friction is sufficient to crack a joint, thus forming another "free end".

On #1 Highway, the characteristics of the concrete, subgrade and dowels caused this initial cracking to occur at intervals of between 40 and 150 feet. If the joints were not sawn in time, or if the slab was weaker elsewhere than at the joint, then uncontrolled cracks formed at about the same spacing. (See plate 10-K).

The concrete then shrinks away from the newly formed joints or cracks, and in order to crack new joints the contraction has to be much greater than before. This is because concrete strength has accrued and therefore a greater subgrade friction must be built up to crack new joints. Thus the cracks that form first tend to be the widest. Temperature effects, naturally, play a part in this behaviour. A drop in temperature at night-time, for example, represents a contraction and is comparable to a certain amount of shrinkage. Probably the prime cause for cracking the narrow joints was the temperature contractions.

After each joint has cracked, successive temperature cycles above the pouring temperature would be expected to equalize the joint width. This did not readily occur on #1 Highway, and suggests that dowel resistance varied from joint to joint, and was weaker at the wide joints.

Most of the expansion and contraction is absorbed by the

wide joints, and the question now in mind is the possibility of dispensing with the narrow joints entirely. The narrow joints, however, relieve stresses and prevent uncontrolled cracks. Thus intermediate joints are certainly desirable in unreinforced slabs. Intermediate joints would not be necessary on reinforced slabs providing the quantity of reinforcement was sufficient to hold cracks together if they should occur.

In several areas, steel less than the total quantity used in the Type A pavements is considered adequate for slabs longer than 30 feet.

For example, the most recent pavement constructed in Ontario* used doweled joints at 99 feet, with steel mesh at seven pounds per square yard. A joint spacing and weight of reinforcement similar to Ontario is used in Illinois, Michigan and Wisconsin, while in New Jersey with this quantity of reinforcement the joint spacing is at 78 feet.[+] In Britain,[x] if the slab length is 80 feet, seven pounds per square yard of reinforcement is specified. The authorities mentioned above place the greater quantity of steel in the longitudinal direction, whereas in the Type A the transverse steel roughly equals the longitudinal steel.

With this type of design, Van Breeman[@] has pointed out that it is imperative to use free moving dowels in the joints. This is

* Private communications with "Stelco" of Hamilton, Ontario, in 1960

\+ Portland Cement Association "Charted Summary of Concrete Road Specifications Used by State Highway Departments in 1958"

x Road Research Laboratory "Guide to Concrete Road Construction", second edition, section two.

@ W.Van Breeman "Experimental Dowel Installation in New Jersey" H.R.B. Proceedings, 1955.

in order to reduce steel stresses if uncontrolled cracks should occur.

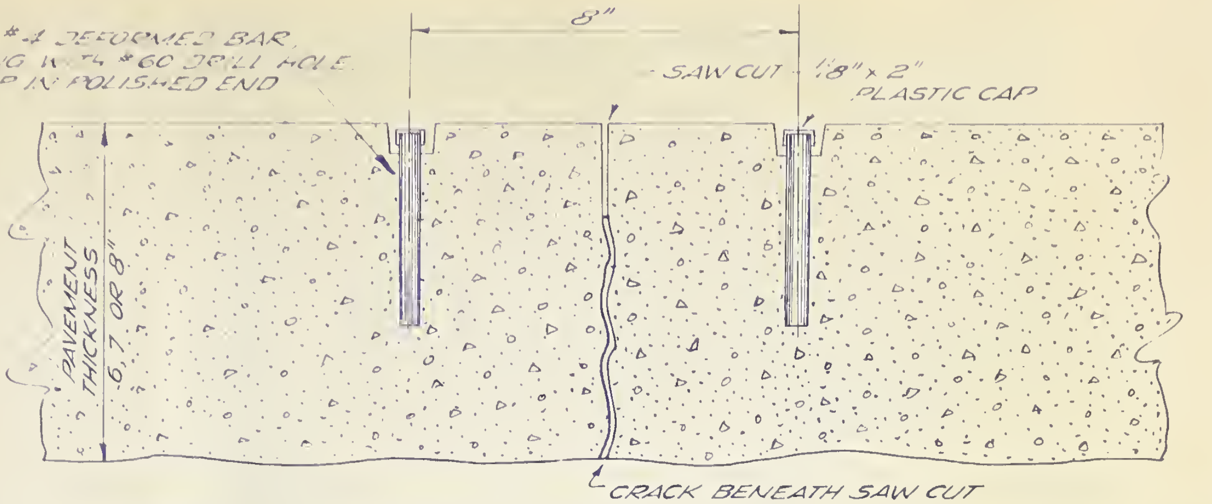
Cashell and Benham* presented data to show that frequent cracking did not occur in slabs of up to 100 feet in length, reinforced with billet steel, and welded fabric, but in slabs over 200 feet cracking frequency was much greater.

* "Experiments with Continuous Reinforcement in Concrete Pavements" H.R.B. Proceedings 1949, figure 23, page 68.

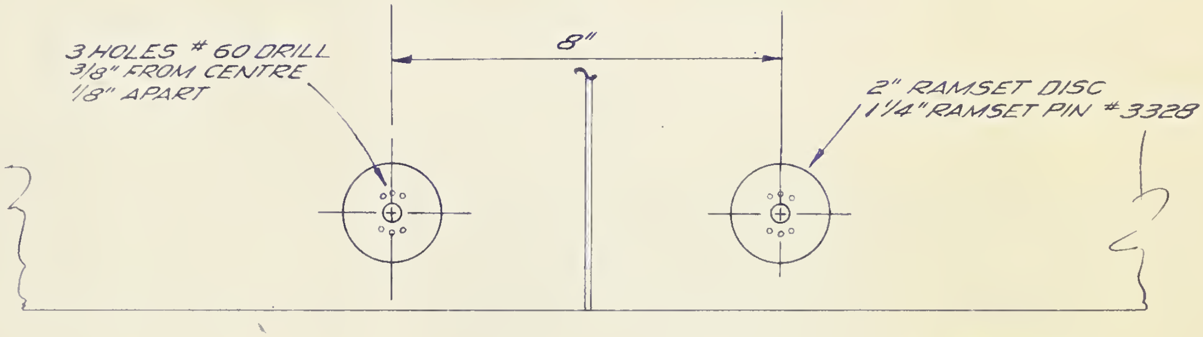
THE MECHANICAL STRAIN GAUGES

PLATE 10-A

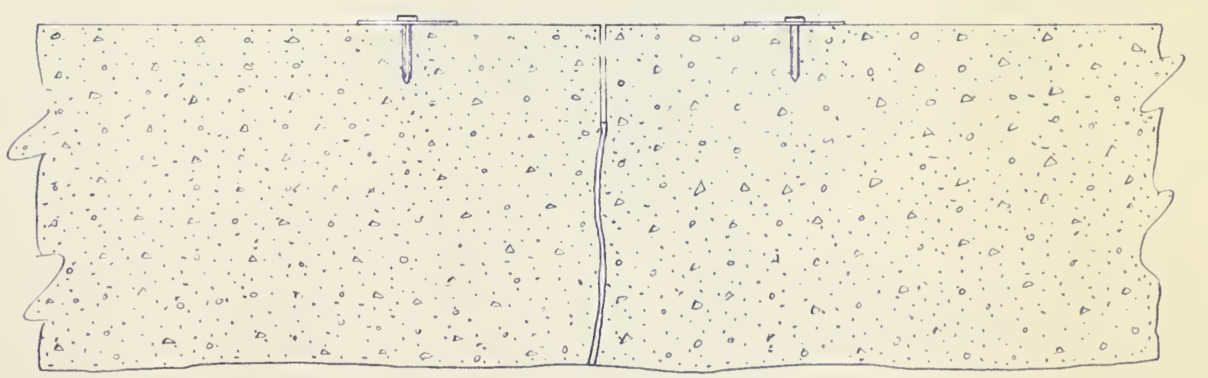
CRACK PINS



TYPICAL RAMSET INSTALLATION



PLAN



SECTION

SCALE: 1/4 FULL SIZE

PLATE 10-B

FIG 1

FIG 2

PLATE 10-C

DEVELOPMENT OF JOINT CRACKING WITH AGE IN AN 8" R.C. ROAD SLAB WITH SAWN DOWELED JOINTS AT 30' INTERVALS

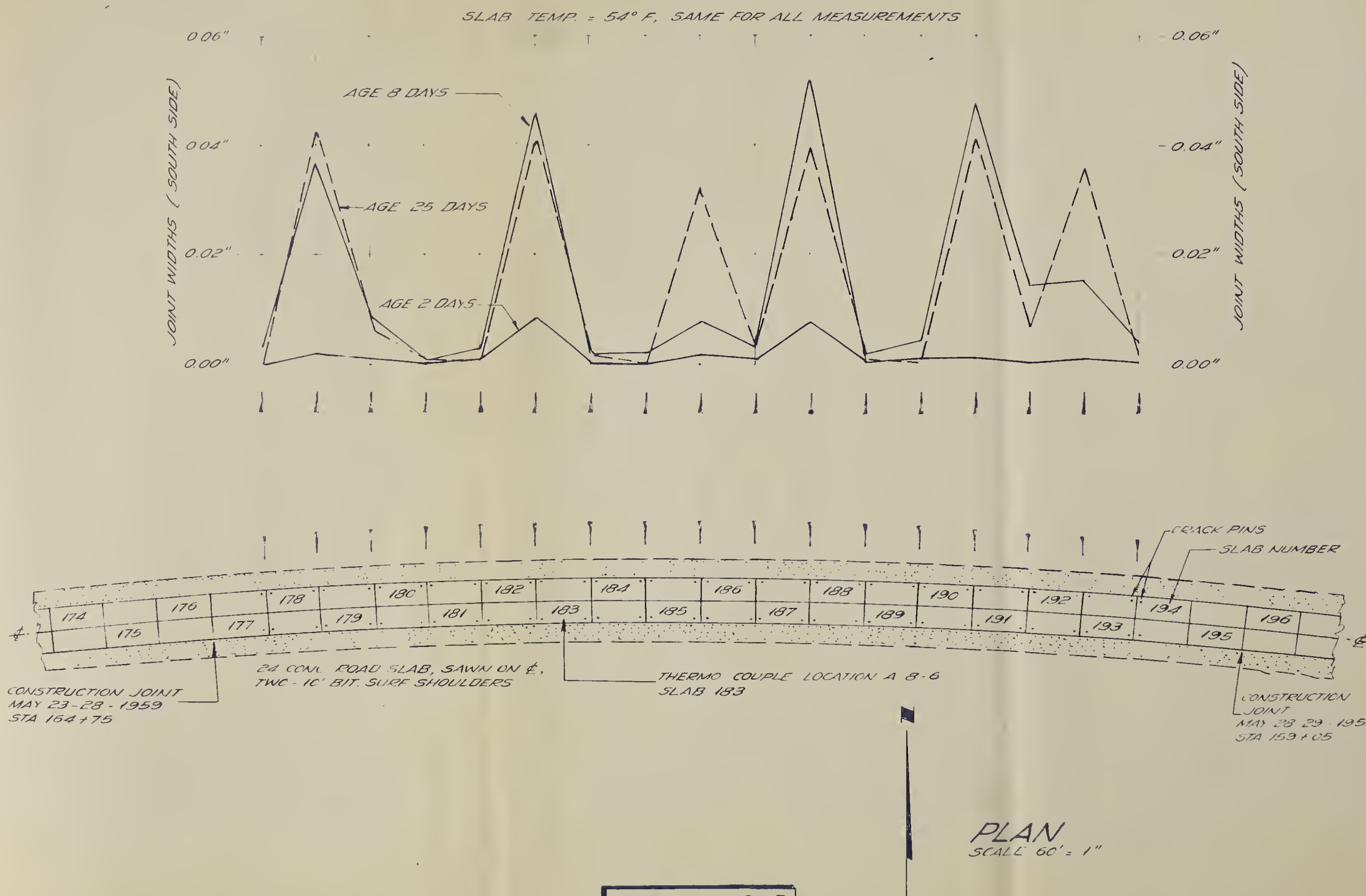


PLATE 10-D

VARIATION OF JOINT WIDTHS WITH TEMPERATURE (FIG. 1) - AND DIFFERENCES IN WIDTH BETWEEN N. & S. SIDES (FIG. 2) - IN AN 8" R.C. ROAD SLAB WITH SAWN DOWELED JOINTS AT 30' INTERVALS

PLAN ON PLATE 7 REFERS

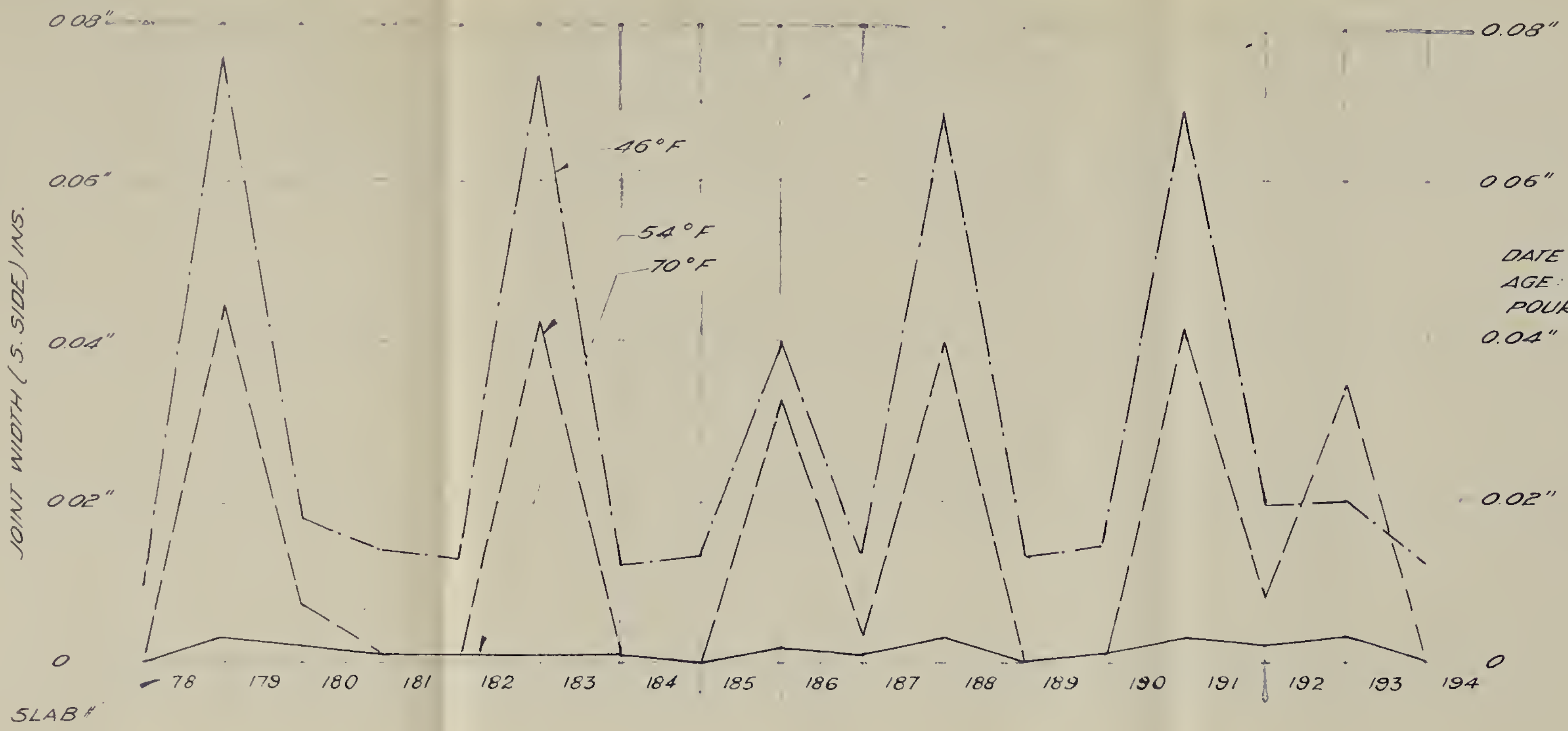


FIG. 1
DATE POURED: MAY 28-'58
AGE: 25-30 DAYS
POURING TEMP: 55°F

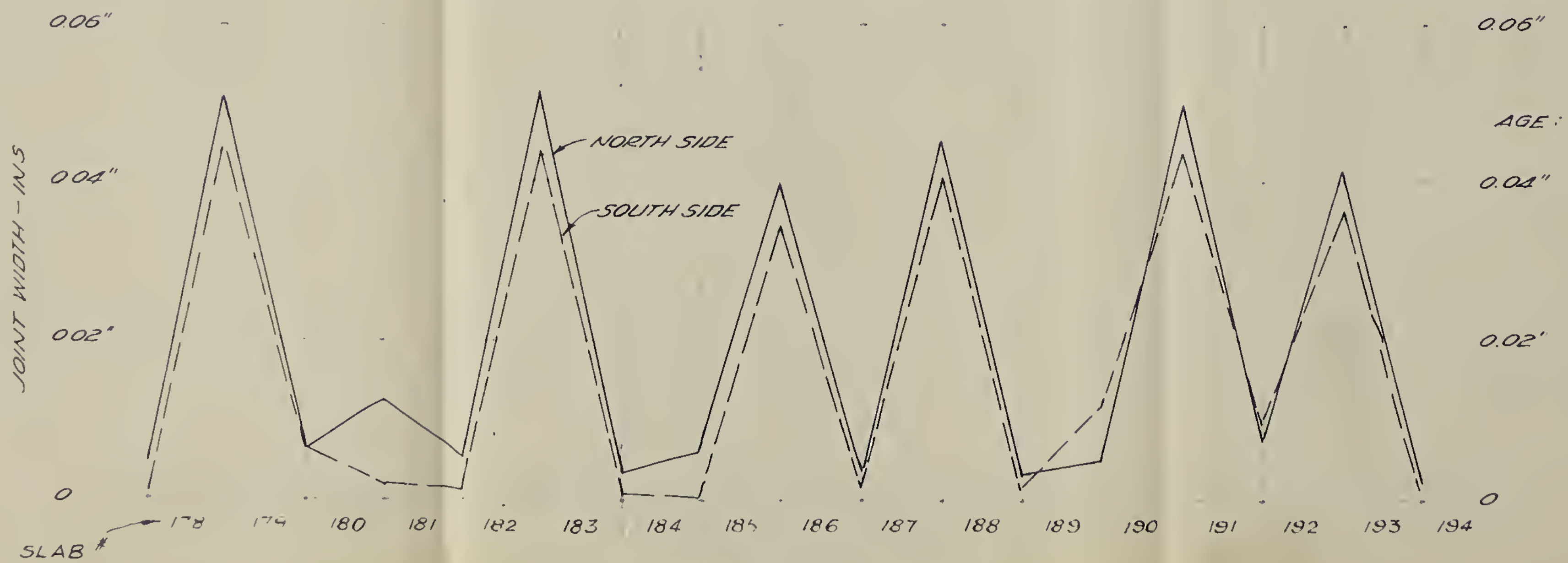


FIG. 2
AGE: 25 DAYS
TEMP 54°F

PLATE 10-E

EFFECT OF HOT WEATHER UPON JOINT WIDTHS (FIG. 1) IN AN 8" R.C. SLAB WITH SAWN DOWELED JOINTS AT 30' INTERVALS

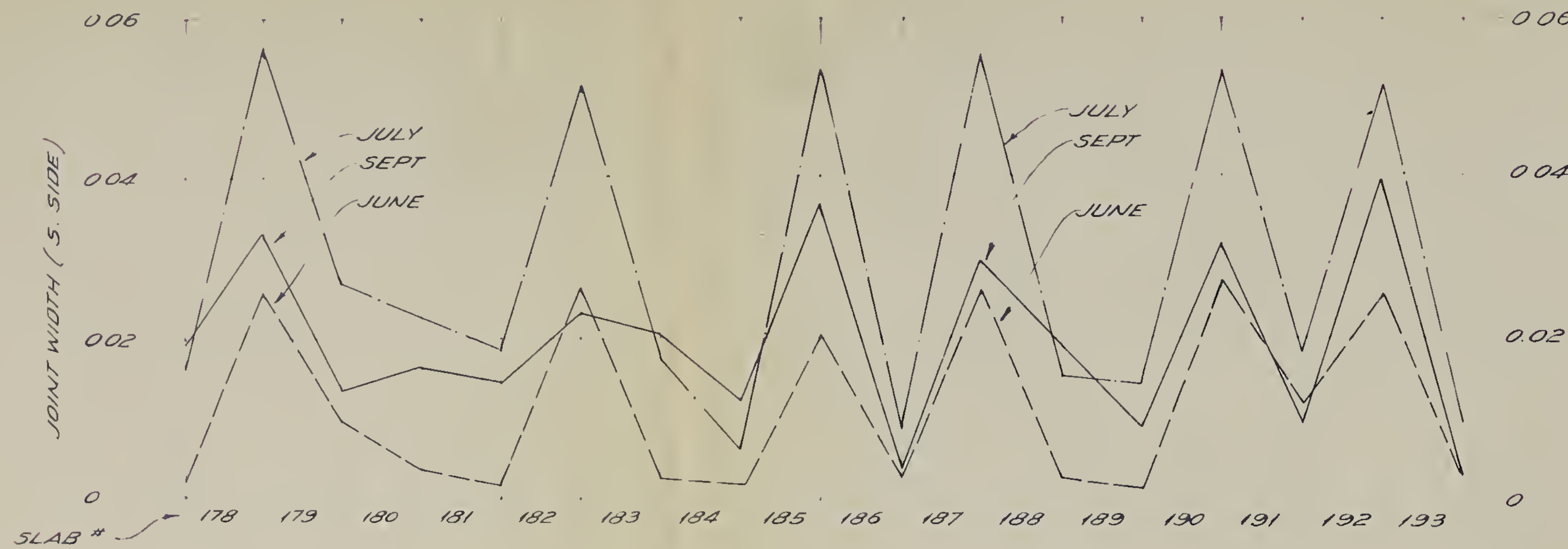


FIG. 1

ALL READINGS AT 56° - 57°F

AGES: JUNE RDG. - 3 WEEKS
JULY RDG - 9 WEEKS
SEPT RDG - 16 WEEKS

DATE POURED MAY 28 - 1958

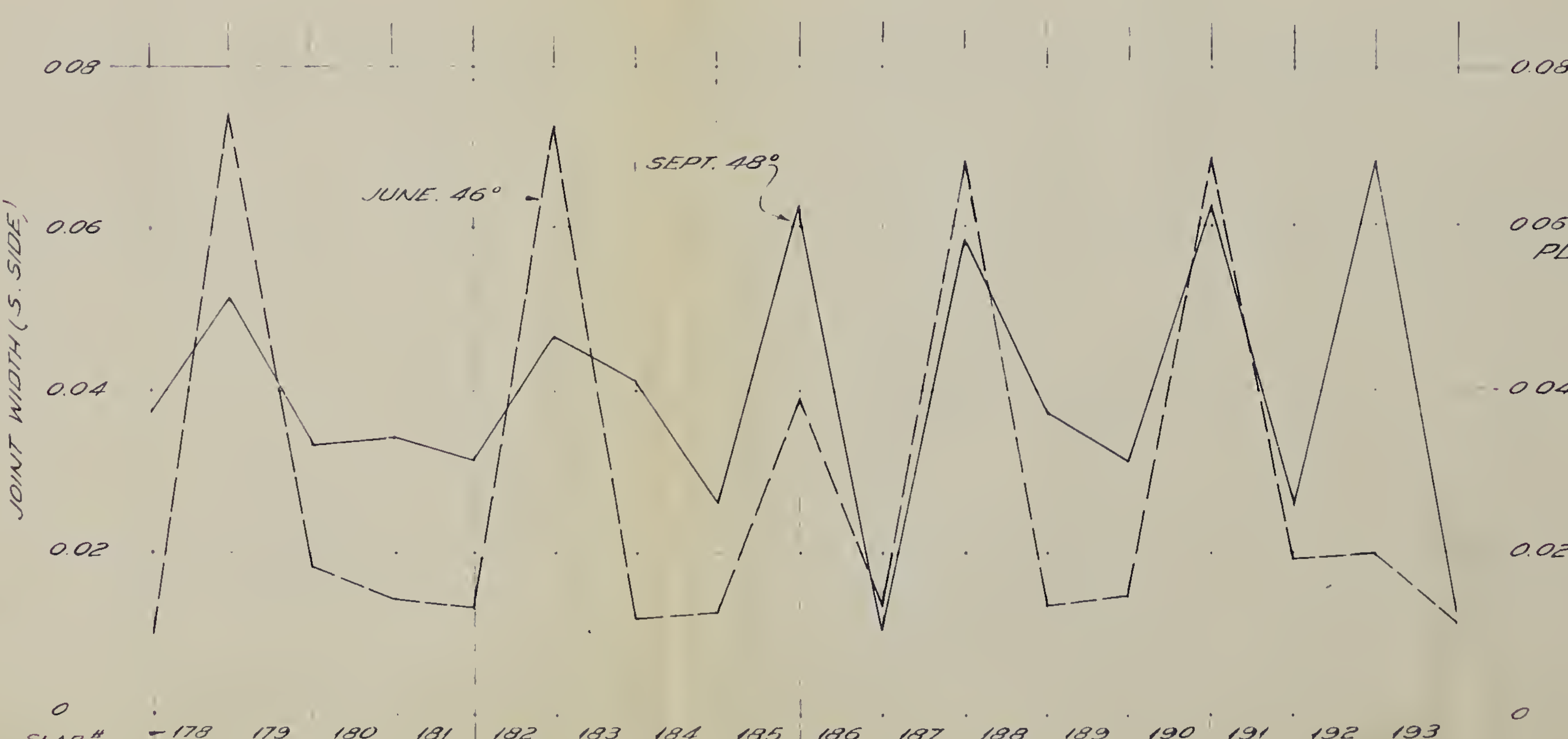


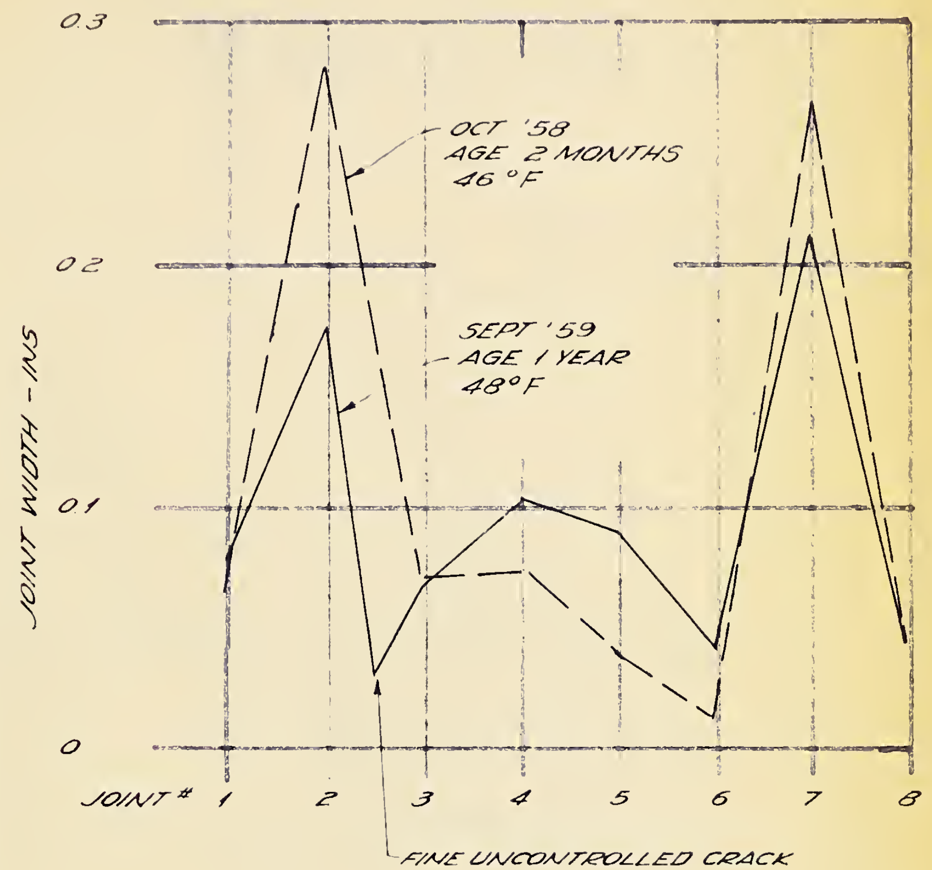
FIG. 2

PLAN ON PLATE 7 REFERS

PLATE 10-F

JOINT WIDTH VARIATIONS OVER 10 MONTH PERIOD FOR AN 8" PLAIN CONC. ROAD SLAB WITH SAWN DOWELED JOINTS AT 20' INTERVALS

LOCATION C-14



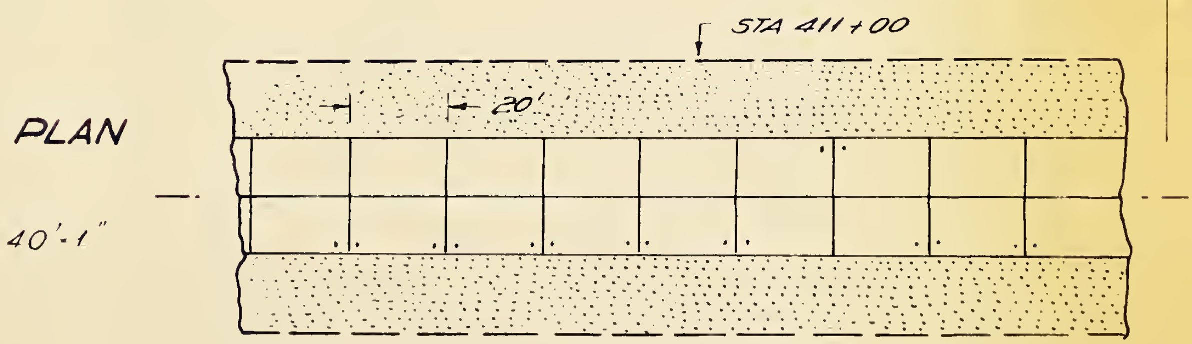


PLATE 10-G

VARIATION IN JOINT WIDTH ON AN 8" R.C. ROAD SLAB WITH SAWN DOWELED JOINTS AT 30' INTERVALS.

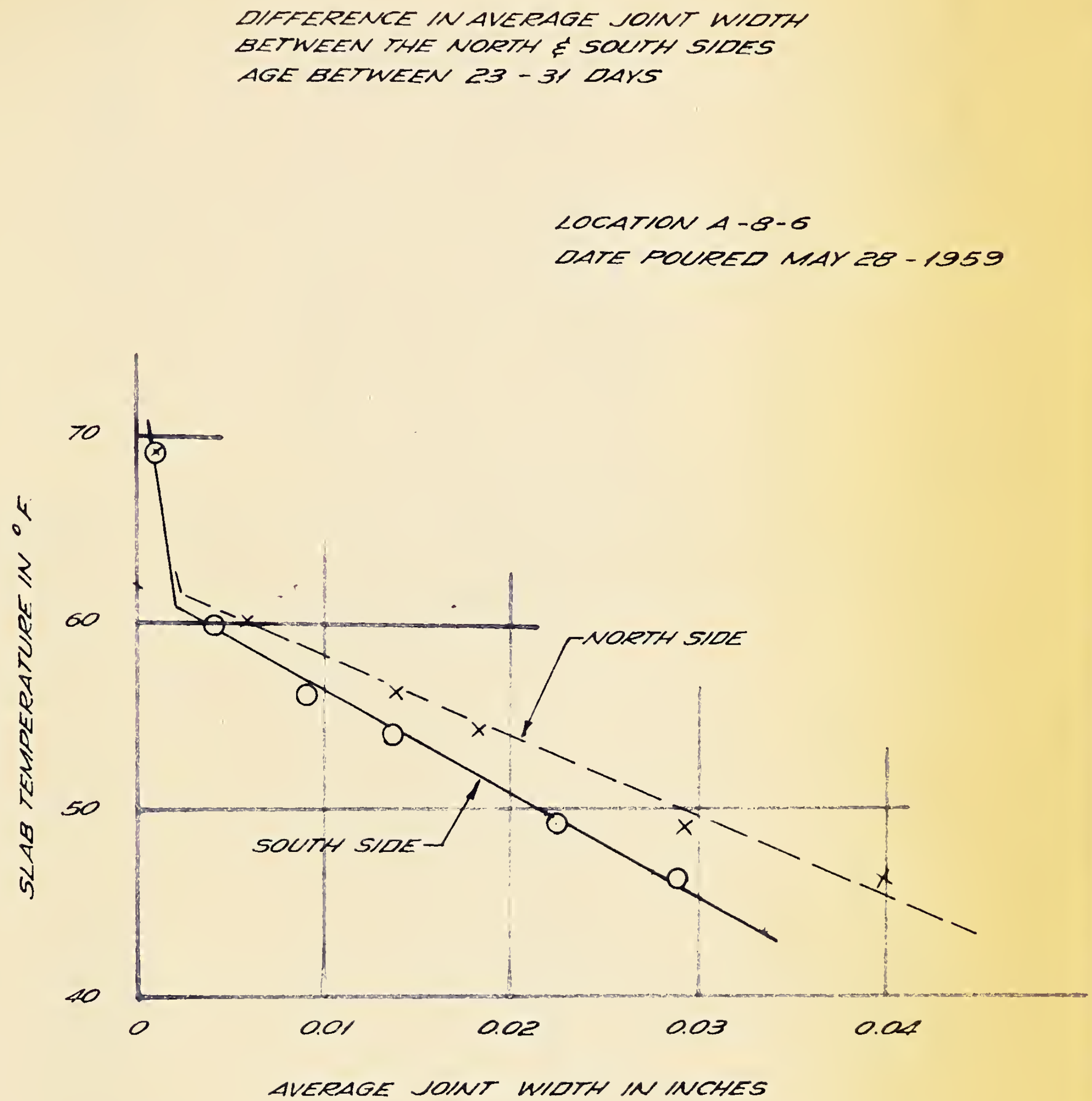


PLATE 10-H

JOINT WIDTH VARIATIONS TYPES C AND A-6

JAN 22 - 1959 TEMP. 4°F AGE 4 MONTHS

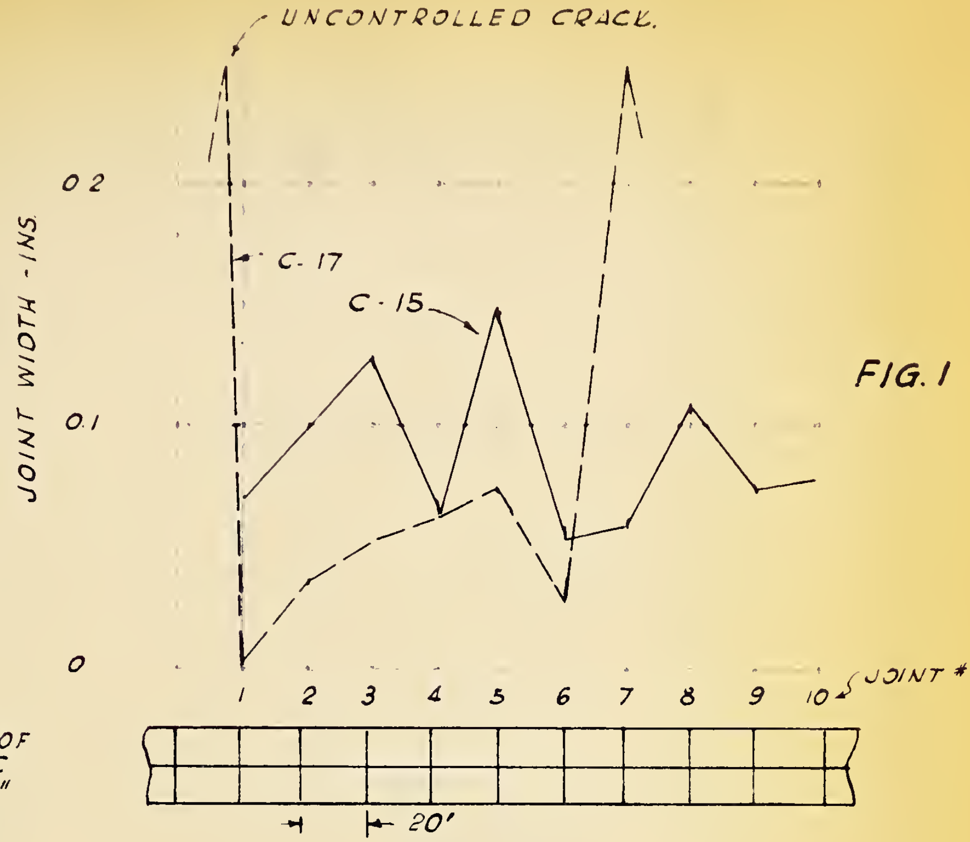


FIG. 1

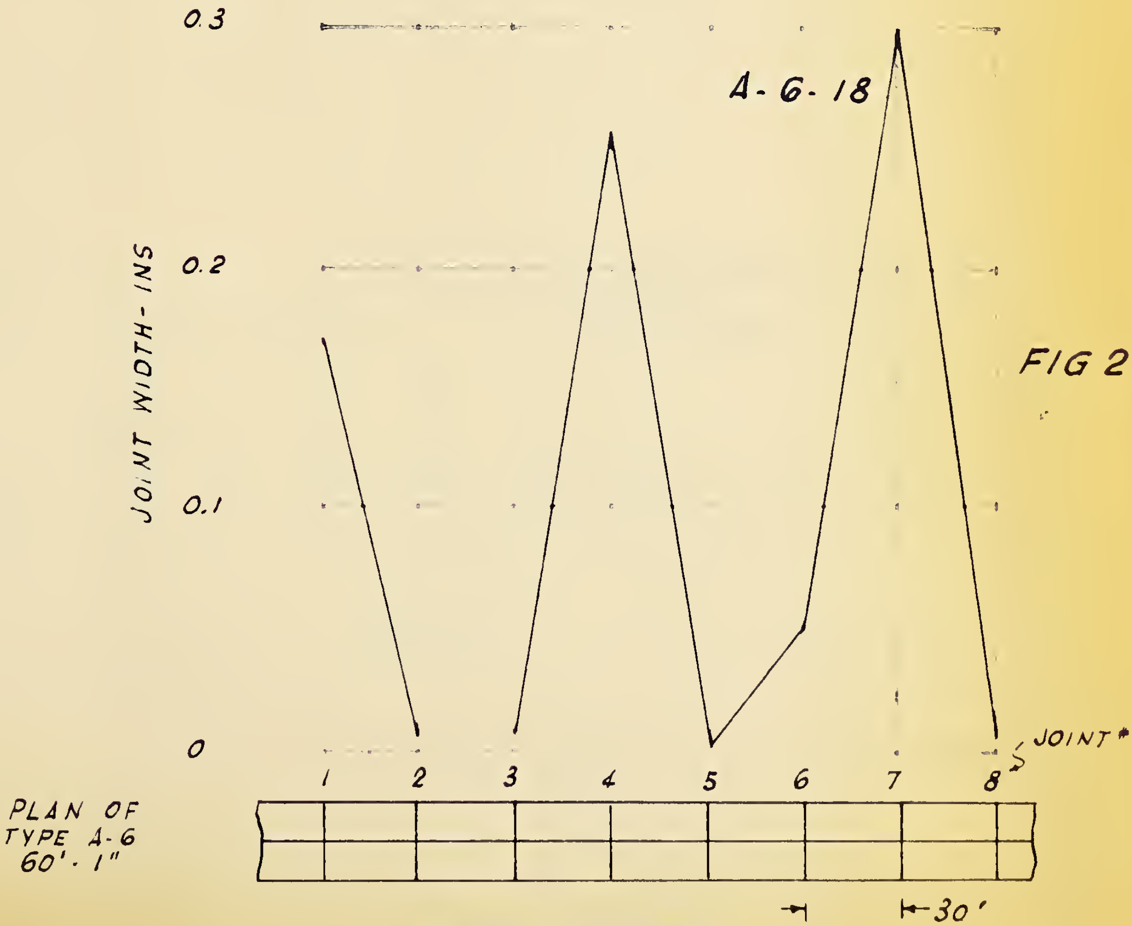
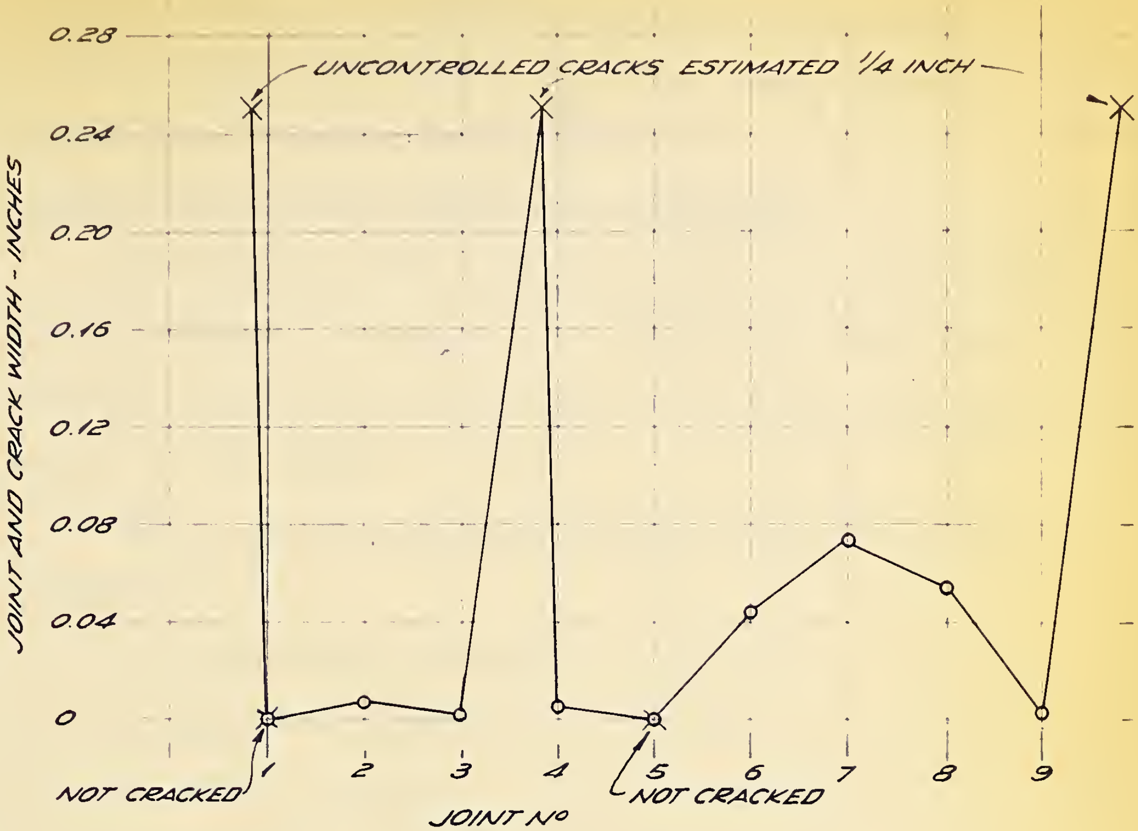


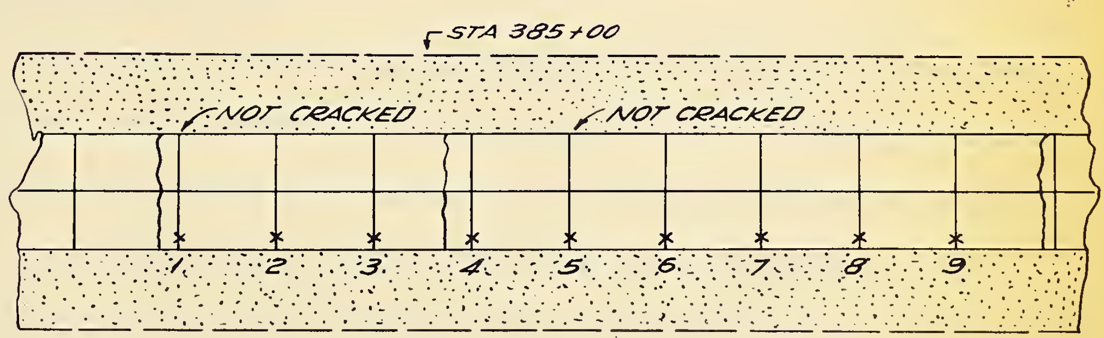
FIG 2

PLATE 10-J

"WIDE" JOINTS REPLACED BY UNCONTROLLED CRACKS IN AN 8" PLAIN CONC. SLAB WITH SAWN DOWELED JOINTS AT 20' INTERVALS

LOCATION C-16 JAN. 22 - 1959
AGE 4 MONTHS
4° F





PLAN

SCALE 40' = 1"

PLATE 10-K

CHAPTER XI AVERAGE JOINT WIDTH VARIATIONS

The first portion of this presentation derives the average joint width versus temperature relationship by analysis, and this leads to a better understanding of the actual field results which follow.

Average Joint Width Variation by Analysis

The average joint width in a series of homogeneous slabs reflects any changes in slab length that occur, and in addition depends upon such factors as dowel seizure or foreign matter in the crack.

Thus, factors affecting average joint width can be summarized as:

1. Temperature changes
2. Subgrade friction
3. Dowel friction
4. Shrinkage effects and creep
5. Foreign matter, such as stones, in the joints
6. Moisture changes

Temperature Effects

Consider a hypothetical series of slabs that do not shrink as they harden, and which rest on a frictionless base. Imagine the joints are formed right through with no dowels.

At temperatures below the pouring temperature, the relationship between average joint width and temperature will be dependent upon the coefficient of linear expansion for the concrete slabs (α). If this coefficient is constant, then the relationship between average joint width and temperature in a long series of slabs of length L_s inches will be:-

Average joint width, in inches, $x = (Tp - T).\alpha.L_s$

T is the slab temperature in degrees Fahrenheit (^oF)

Tp is the pouring temperature of the mix in ^oF

is in inches/inch/^oF

Instead of Tp, a more correct value would be Tp + tc ^oF,* the temperature at which the concrete transforms from a plastic state to a solid state. Because of heat generated as the concrete hardens, and because of the sun's heat, the temperature tc is usually positive.

At temperatures greater than Tp + tc, the relationship should be x = 0.

Shrinkage Effects

If shrinkage within the concrete occurs, the effect will be to increase the average crack width by an amount, say x_s inches. This effect can be represented as an increase in the value of the intercept on the temperature axis by an amount, say ts, where $xs = ts.\alpha.Ls$

Therefore, $x = (To - T).\alpha.L_s$, where To = Tp + tc + ts

This equation can be written in the form $T = To - \frac{x}{\alpha Ls}$ and Ls is termed "the temperature susceptibility of the joint." At temperatures greater than To, the relationship should again be x = 0.

Thus, the net effect of initial shrinkage, and the heat generated during hardening upon readings taken after shrinkage is completed, is to produce an intercept To, higher than Tp on the temperature axis.

Any additional shrinkage effects, such as creep, would cause a further increase of the temperature intercept to To_2, say.

* Lower case letters refer to incremental temperatures.

Subgrade Friction

Subgrade friction will tend to increase the crack width if temperature is rising, and the opposite will occur for falling temperatures. Hence, at any particular temperature, the crack width can show a range of values.

Dowel Friction

If it is assumed that the resistance of the dowels is of a simple, frictional nature, dowel friction will act in the same manner as subgrade friction.

Thus, if respective values of temperature and average crack width are plotted, the net result of subgrade friction and dowel resistance will be to cause a scatter of the experimental points.

The most consistent values of crack width and slab temperature are likely to be obtained when the slab is neither expanding nor contracting.

Foreign Matter in the Joints

The effect of foreign matter, such as stones, in the joints, or severe dowel rusting, would be to prevent complete joint closure. Only the relationship at temperatures above To will be affected, and the vertical trend of the graph should occur at $x = C$ instead of $x = 0$ (c is the effective width of the foreign matter.)

Moisture Effects

An extract taken from "Concrete Pavements" by the Road Research Laboratory in England* states that midway down a road slab the free moisture was never less than its initial values

* paragraph 9.50

of 4.6 per cent. The surface moisture varied with a minimum of 3.2 per cent. This particular reference then goes on to state that "little information is available about the distribution and changes of moisture content, but approximate results obtained at the Road Research Laboratory indicate that the movements produced are small and much less than those due to temperature effects." The rainfall in Britain is fairly uniformly distributed throughout the year.

For climates in the U.S.A. that are damp in winter and dry for long periods in the summer, the Highway Research Board Committee of Joint Material in Concrete Pavements* states "The overall seasonal changes in joint width are significantly reduced by the effect of seasonal change in moisture content."

At Calgary, precipitation occurs on the average during every month of the year, but is highest in the summer. The average rainfall between May and August is 2½ inches per month.+

Summary of the Theoretical Analysis

The joint width versus temperature variations that would be expected from the analysis above are shown on plate 11-A. From the slope of the line below To, the average value of α can be found, i.e. $\alpha = \frac{\Delta x}{\Delta T} \cdot \frac{1}{L_s}$

Field Procedure

A general idea of the methods of measurement has been given

* "Filling and Sealing Joints and Cracks in Concrete Pavements" H.R.B. Bulletin 78, page 12, 1953.

\+ Department of Transport and National Research Council "Climatological Atlas"

previously and are now discussed in more detail.

Joint width measurements were made on crack pins at the surface of the slabs. It was found that the most consistent results were obtained when the overall temperature gradient within the slab was less than $4^{\circ}F$ over a depth of 8 inches, and that the best temperature for plotting was the temperature at mid depth. Slab temperatures were likely to be closest to uniform and unchanging for an hour or so at sunrise. Sometimes, also, good measuring conditions might prevail in the late afternoon in sunny weather, and under heavy cloud in cooler weather.

Slab temperatures were recorded at one inch from the surface, one inch from the bottom, and at mid depth of the slab, before and after taking each set of joint width readings.

The accuracy of the slab temperature measurements depended upon the accuracy of measuring the cold junction temperature. The most suitable thermometer that was available, read to the nearest $\frac{1}{2}^{\circ}$ Centigrade, i.e. about $1^{\circ}F$. Thus it was decided to keep a mixture of ice and water in the thermos bottle, which then would keep the cold junction always at $32^{\circ}F$.

The potentiometer enabled the temperature to be read with an error not likely to exceed $\frac{1}{2}^{\circ}F$, on the 1959 pavements, and $1^{\circ}F$ on the 1958 pavements. (The pavements laid in 1958 had single thermocouple wires but in 1959 these were doubled to improve accuracy.) All temperatures are therefore equal to the nearest $1^{\circ}F$.

For a wide joint, a change of $1^{\circ}F$ would be equivalent to a change in width of about 0.0005 inches.* The "Soiltest"

* See plate 12-B

mechanical strain gauge could read slightly more accurately than this, while the "Berry" could be read slightly less accurately. Crack width data was therefore recorded to the nearest 0.001 inches, which was compatible with temperature readings to the nearest 1^{o}F.

Discussion of Results for Location A-8-6

Data for location A-8-6 is presented in plate 11-B, and a plan is given in plate 10-D.

It was decided to average all seventeen joints to obtain the average crack width, since the ratio of narrow joints to wide joints on this test strip seemed representative for the type of pavement.

As far as possible, points for plotting have been selected from data in which the temperature change during the set of readings was not greater than 2^{o} at the slab centre, and the temperature gradient from top to the middle not greater than 2^{o}F.

The results were plotted so that the largest possible temperature range was covered in a few days. This was to eliminate as much as possible time variable effects such as creep.

Results for June shown as a full line on the graphs, cover the period between the end of initial shrinkage and the onset of summer creep.

The pouring temperature of the mix was 55^{o}F, while the extrapolation of the graph to cut the temperature axis gives To = 63^{o}F. Thus, Tc + Ts was 8^{o} and equivalent to about 0.016 inches of average crack width. This represents a

shrinkage of 44×10^{-6} inches/inch of concrete, at the surface of the slab.

The average slab length was 29.8 feet on the south side and 30.1 feet on the north side. Hence, from the slope of these graphs, $\alpha = 6.3 \times 10^{-6}$ inches/inch/° north side and 4.8×10^{-6} south side.

Above 62°, according to the theory, the crack widths should read zero. The slight slope obtained could be attributed to the following causes:

1. The joint cracks were wider at the surface than at the bottom. This could be because the sun's heat caused a differential temperature gradient while the concrete was changing from plastic to solid state, and in addition greater shrinkage effect was likely at the surface. Thus, under the fairly uniform temperature gradient used for subsequent measuring, the bottom of the "average" crack would close at about 62°, while the top would not close until a much higher temperature.
2. The 8 inch gauge length of concrete itself was being compressed by the slabs on each side. The unit strain in this gauge length should therefore be $\alpha(T-To)$, and the slope of the line should be 2×10^4 °F/inch. The slope is actually about 0.88×10^4 (i.e. flatter). Thus, the greater part of the explanation must be attributed to the first reason, namely crack widths wider at the surface than at the bottom. The negative values of crack width are probably, however, the result of compression.

In July, the creep caused To to rise to 74°, and also a slight reduction in α on the north side. It is felt, however,

that this was a temporary effect, because in September the summer creep was partially recovered, and the graph slope closer to its original value. In September α had values of 5.4×10^{-6} south side and 5.7×10^{-6} north side.

It appears that in June and July there was no value for c, i.e. no appreciable quantity of foreign matter in the joints.

Some limited confirmation of the minor effects of moisture, as surmised by the Road Research Laboratory, England, was obtained. Prior to the reading taken at 46° in the period June 19th to 28th, the pavement had been wet for about twenty-four hours. This reading, however, follows the same graph as all previous readings taken under dryer conditions.

Pavement type C

Data for location C-14 is presented on plate 11-C, and a plan is given on plate 10-C.

In this stretch of concrete there was one wide crack for about every four narrow cracks. Hence joints 4 to 8 inclusive were averaged.

The results reproduce the effects described for location A-8-6, but differ in detail as described below.

In July, To was 81°, and therefore the effect of "weather creep" was more intense at this location than at A-8-6, probably because the slabs are unreinforced. To, with type A-8, was only 74° under these weather conditions, but originally after pouring had been the same for both types.

There are indications also of foreign matter within the joints. Even at 90° the average crack width was 0.01 inches, and it can be estimated that c is about 0.02 inches since graph

slope changed at about this value. It is quite probable that foreign material entered when the joints were wide in winter 1958-59. The slope of the upper portion of the graph is less steep than for type A-8. This could mean that foreign matter lodged in the joint cracks exists in small pieces, and is thus easily compressed. Alternatively the crack widths at the surface might have been wider than at location A-8-6, because of greater temperature differential during pouring, or greater surface shrinkage.

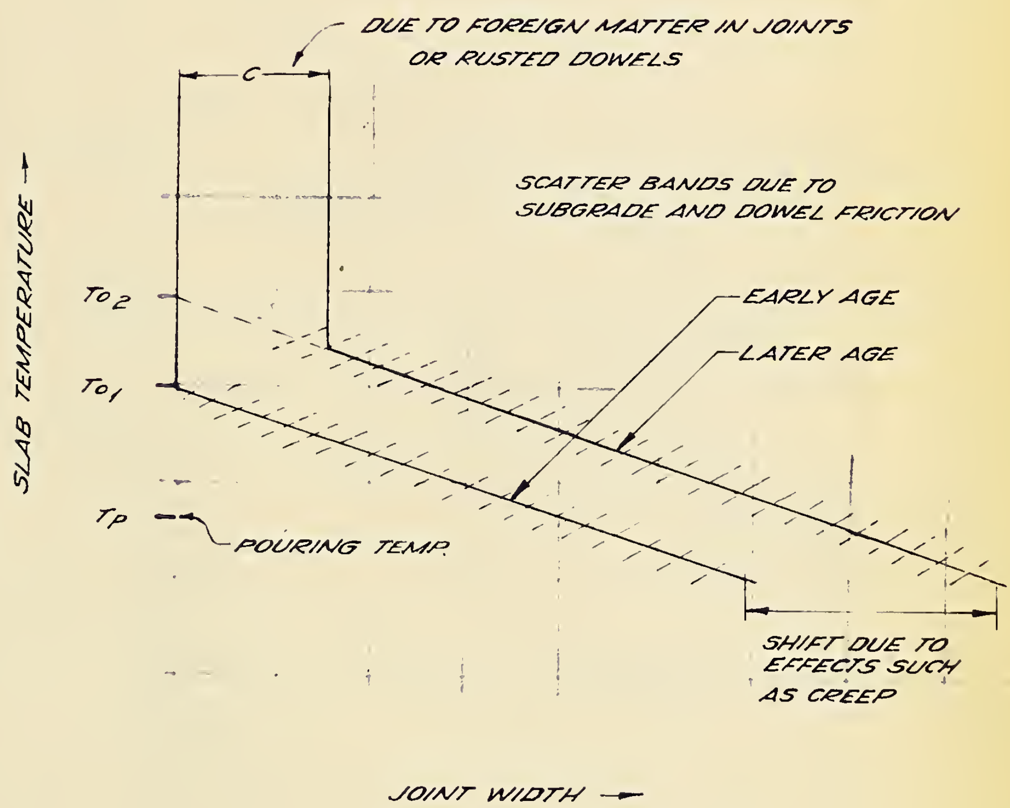
With a slab length of 20 feet, α is 6.6×10^{-6}, and is about the same for each time of the year measurements were taken.

Pavement Type A-6

A plan of the location, location 18, is given on plate 10-J, and the results on plate 11-D.

The results obtained are very similar to those of Type C. Since the air temperature was lower during pouring, the value of To is lower. With a slab length of 30 feet, α is 5.9×10^{-6}

HYPOTHETICAL CRACK WIDTH/TEMP VARIATIONS



$T_{o1} - T_p = t_c + t_s$ - DUE TO INITIAL SHRINKAGE AND HEAT GENERATED PRIOR TO HARDENING

PLATE 11-A

VARIATIONS IN AVERAGE JOINT WIDTH ON AN 8" R.C. ROAD SLAB WITH SAWN DOWELED JOINTS AT 30' INTERVALS

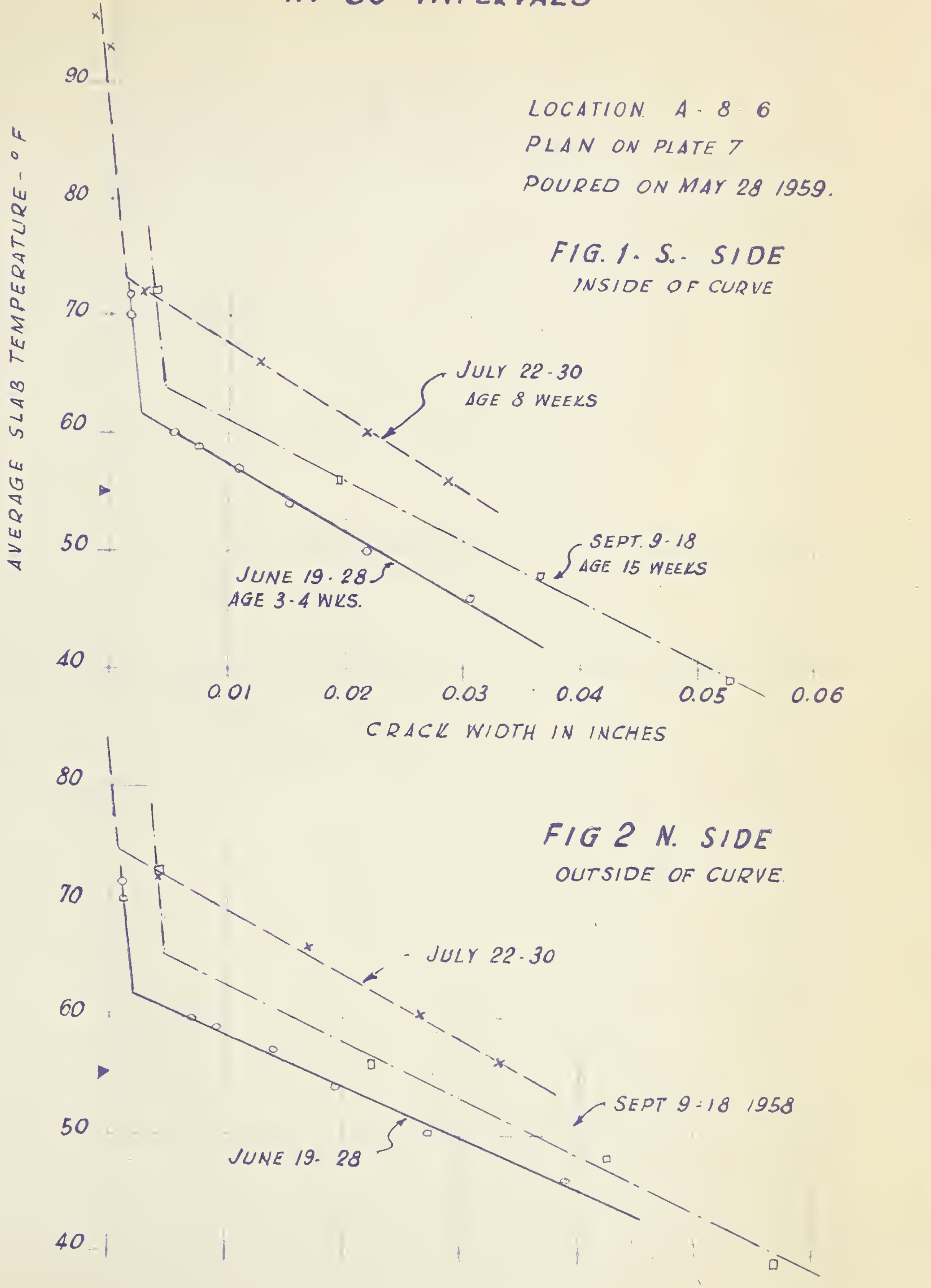


PLATE 11-B

VARIATIONS IN AVERAGE JOINT WIDTH ON AN 8" PLAIN CONCRETE ROAD SLAB, WITH SAWN DOWELED JOINTS AT 20' INTERVALS

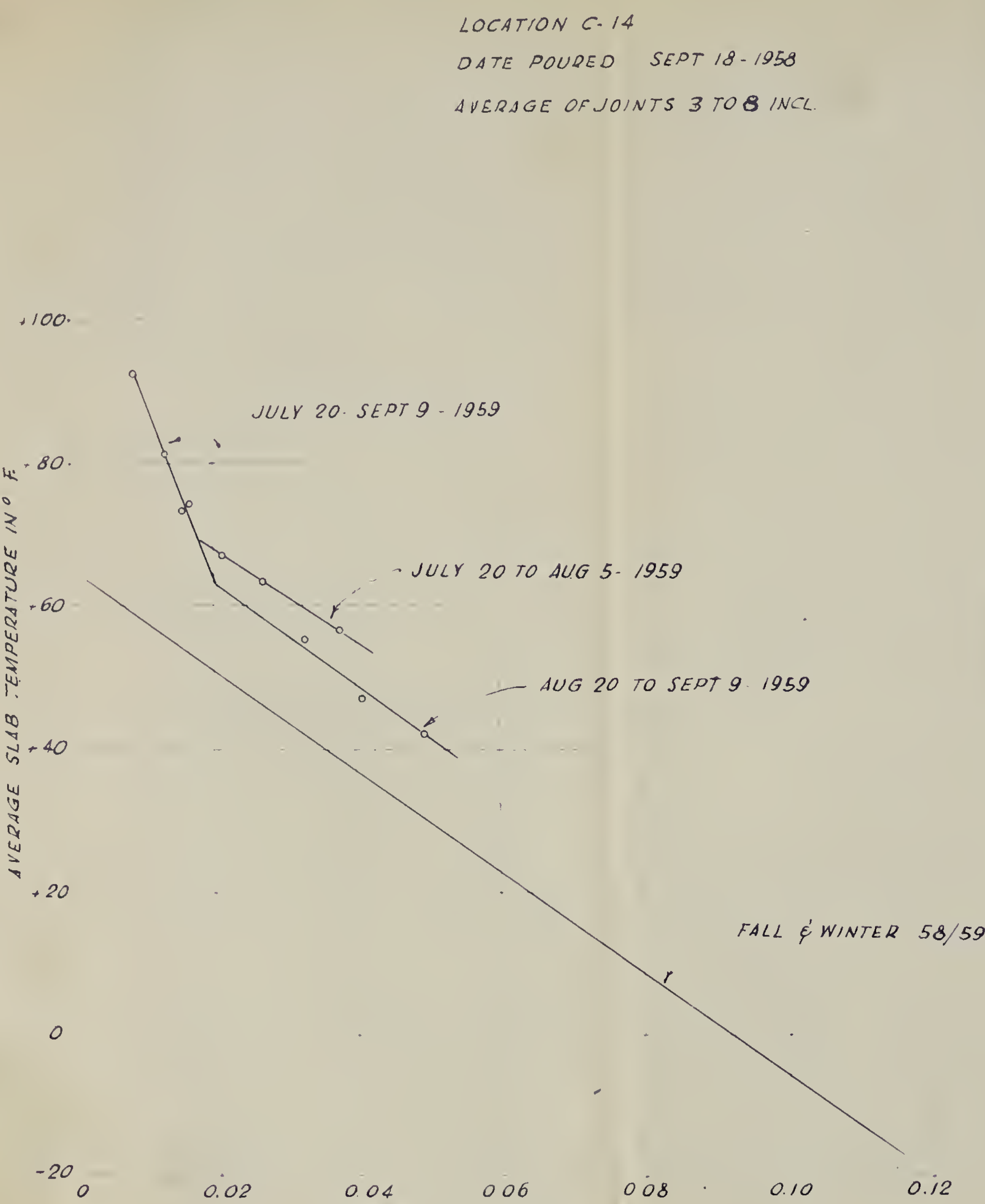


PLATE 11-C

VARIATION IN AVERAGE JOINT WIDTH ON A 6" R.C. ROAD SLAB WITH SAWN DOWELLED JOINTS AT 30' INTERVALS

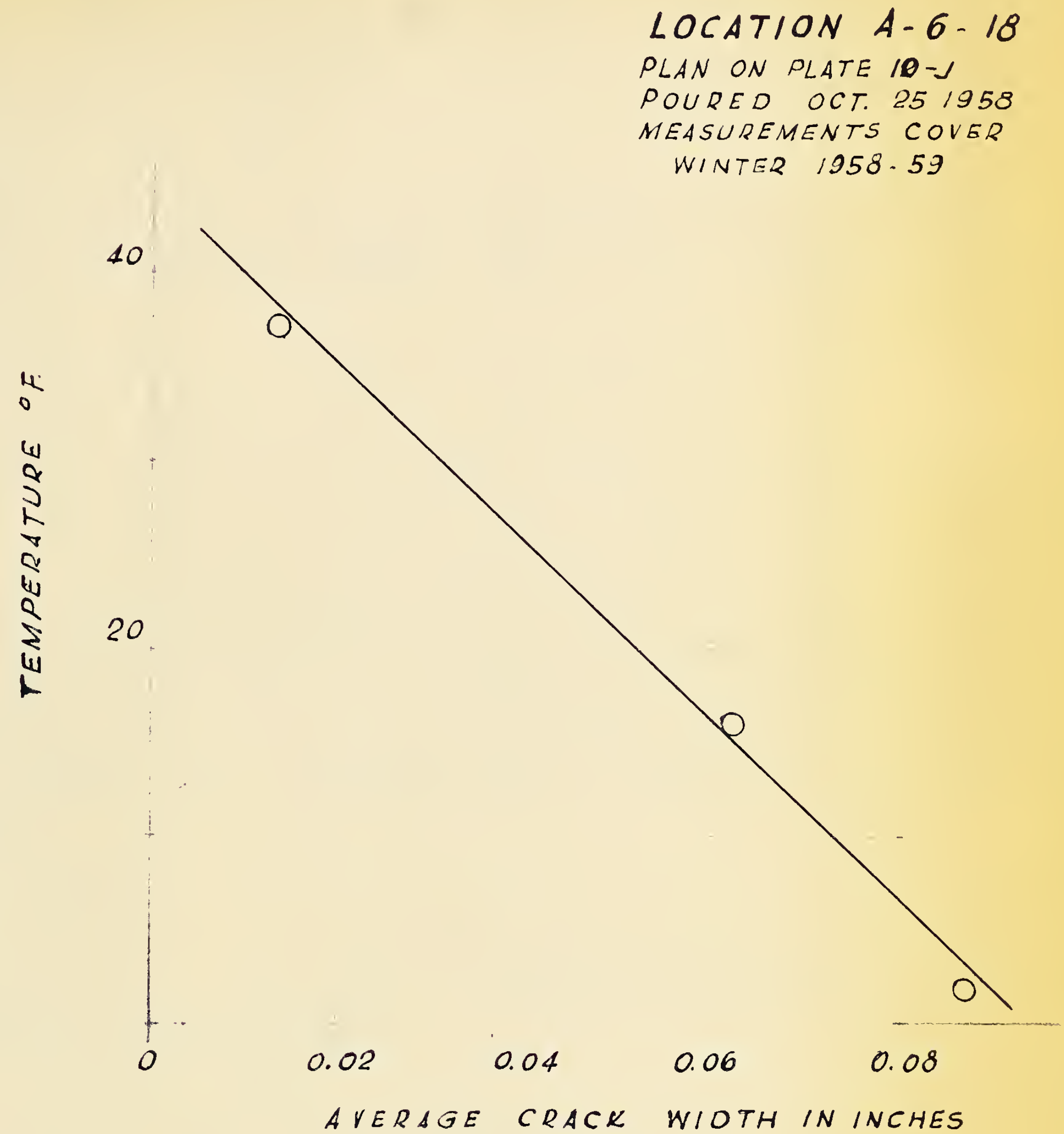


PLATE 11-D

CHAPTER XII TEMPERATURE SUSCEPTIBILITY OF WIDE AND NARROW JOINTS COMPARED

Location A-8-6

On plate 12-A a typical wide joint and a typical narrow joint are compared with the average joint width at this location. A plan is given on plate 10-D, and the two joints compared are 189 - 190 and 190 - 191.

The results show that the relationship between the wide, narrow and average joint widths, below the To temperature can be expressed as:

W_w or $W_n \pm$ constant width = Fixed Ratio x X

where W_w = width of wide joint at any particular temperature

W_n = width of narrow joint at any particular temperature

X = average joint width at any particular temperature

In the early part of the summer, soon after pouring, plate 12-A figure 1, shows that the relationship between the wide joint and the average joint at any particular temperature below 63°F was given by:

$$W_w = 2.35\ X$$

In the fall, an examination of plate 12-A, figure 2, shows that the relationship became:

$$W_w - 0.013'' = 1.28\ X$$

The relationship for the narrow joints was

$W_n + 0.005'' = 0.5\ X$ in the spring, and

$W_n = 0.7\ X$ in the fall

Comparison of other joints will likely give slightly different ratios. But the equations above do show that the wide joints were absorbing a greater percentage of the thermal movement

of the slabs in the spring, than in the fall.

The data presented above illustrates in a more quantitative manner the tendency of the hot summer weather to equalize joint widths. The stiff dowels on the narrow joints must tend to free up a little.

Location C-14

Data is presented on plate 12-B while the plan is on plate 10-G. Joints #6 and #7 are compared.

In the first winter after pouring, the joint relationship was given by:

$$Wn = 3.0\ X$$

while in the fall of the following year the ratio had reduced to:

$$Wn = 1.9\ X$$

The ratio of narrow joint width to average joint width, showed an increase from about 0.2 in the first winter to 0.4 the following fall.

Thus, the concrete poured in the fall of 1958 tended to produce greater differential widths than the concrete poured in the spring of 1959. This is unlikely to be due to temperature differences because the daily temperature range in the first few days after pouring was about the same at both C-14 and A-8-6 (between 60° and 34°). It could be that dowel placement in 1959 was better than in 1958, or that the difference in mix design had some effect.

The results for the wide joint at C-14 show that even at very high temperatures, it was not completely closed. This indicates that foreign matter occupies roughly 0.04 inches of the joint width.

Location D

Since the slope of the joint width/temperature relationship probably reflects the effect of dowel resistance at that joint, it was decided to examine for comparison a series of joints on #2 Highway. These joints have no dowels.

Ramset discs were used, and by plotting strain gauge readings against temperature it was hoped that the initial readings could be estimated from the readings at change of slope.

Unfortunately, although the temperature rose to over 80°, no change in slope was noticed. Hence initial readings and actual joint widths could not be estimated.

On plate 12-C are plotted gauge readings versus temperature for seven consecutive joints at this location. The interesting feature is that all slopes are about the same, and give a value of about 6.3×10^{-6} for . The characteristic differences in slope between various joints as obtained on #1 Highway do not appear on #2 Highway.

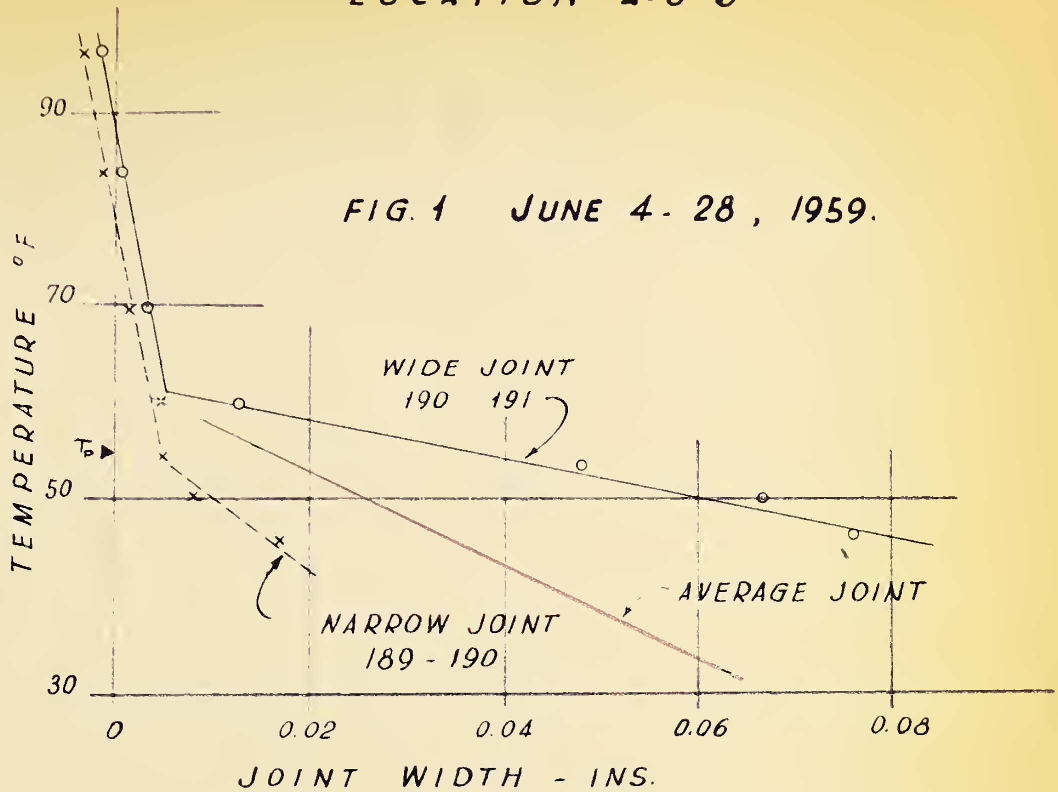
This provides further evidence that the system of wide joints and narrow joints is the result of uneven dowel resistance in the joints.

Discussion of Results

The factor which tends to equalize the joint widths is the creep of the concrete under compressive stresses. The results presented seem to indicate that uneven joint widths are maintained because of varying dowel resistance from joint to joint. Thus, the system of "units", although initiated by shrinkage characteristics, is maintained by the variations in dowel resistance between the wide joints and the narrow joints.

TEMPERATURE SUSCEPTIBILTY OF WIDE JOINTS AND NARROW JOINTS COMPARED

LOCATION A-8-6



NOTE: SUSCEPTIBILITY OF AVERAGE JOINT THE SAME IN BOTH CASES.

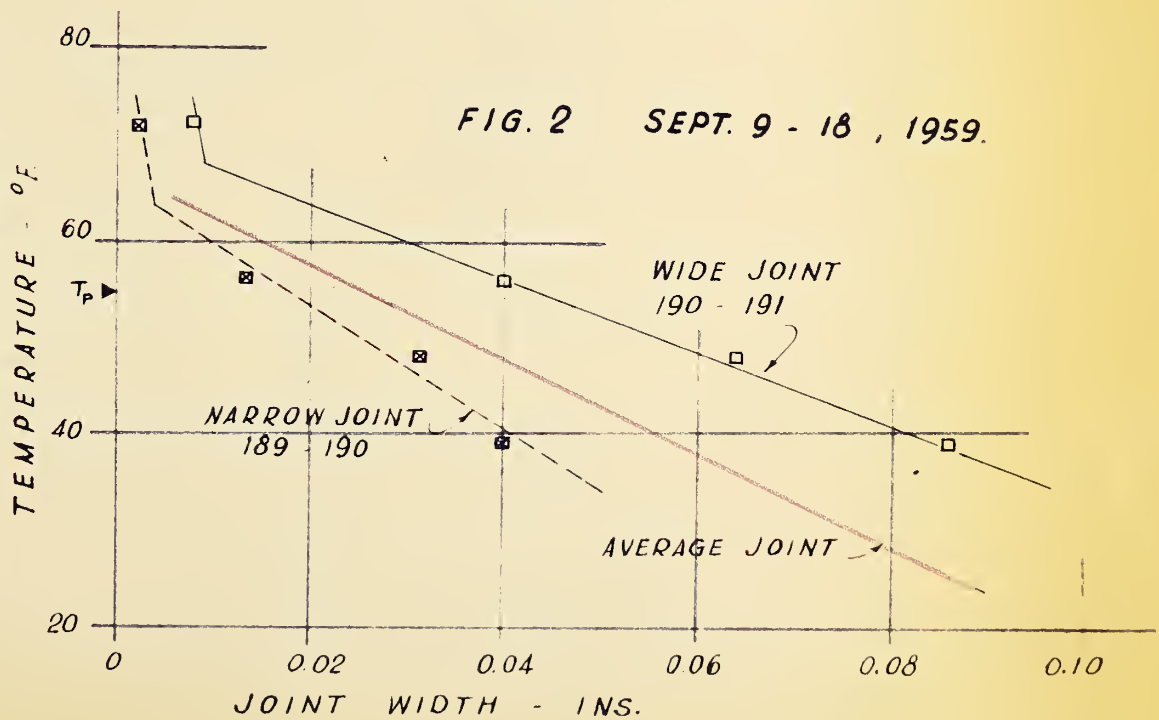


PLATE 12-A

TEMPERATURE SUSCEPTIBILITY OF WIDE AND NARROW JOINTS COMPARED

LOCATION C-14

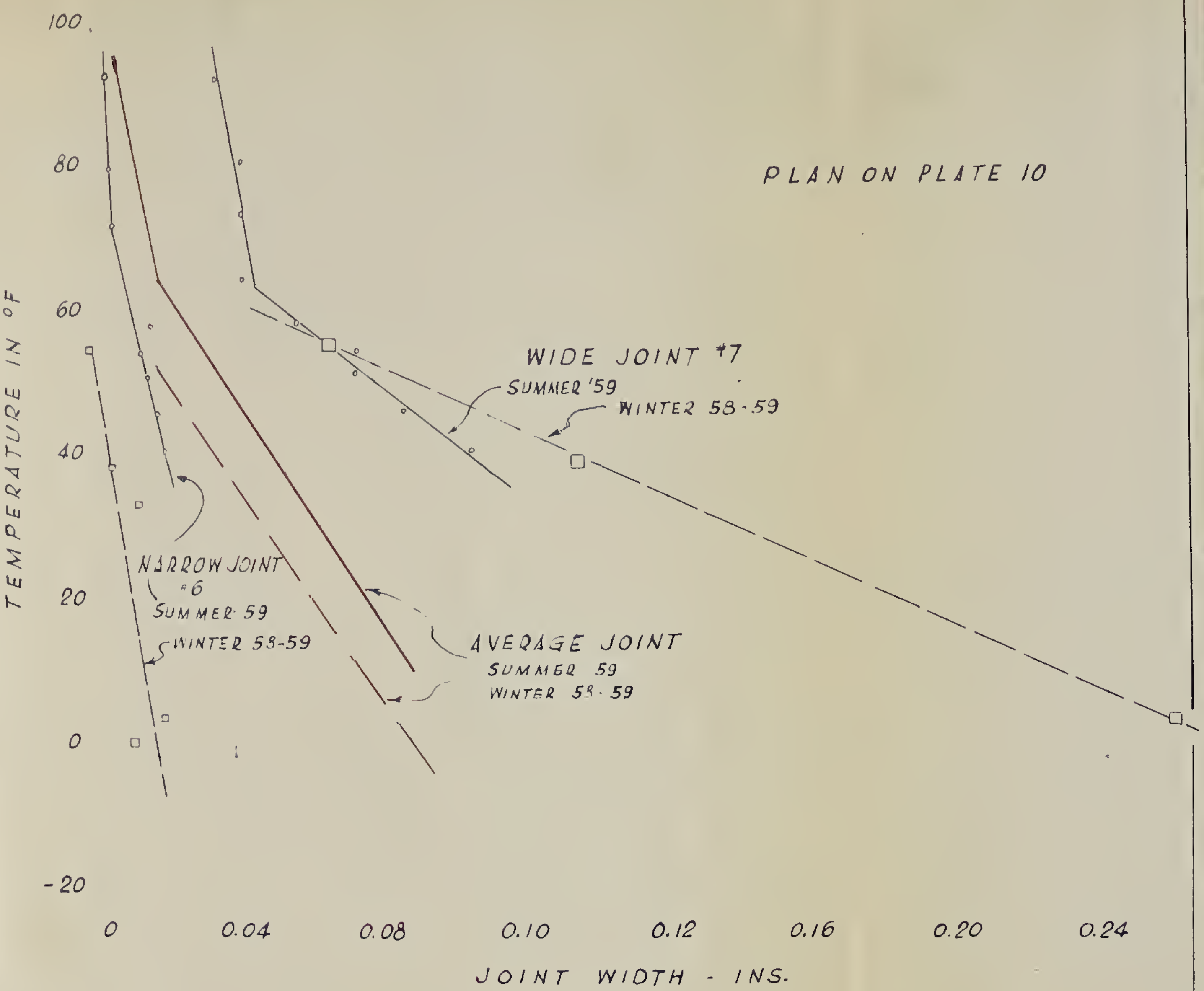


PLATE 12-B

TEMPERATURE SUSCEPTIBILITY OF DOWELESS JOINTS

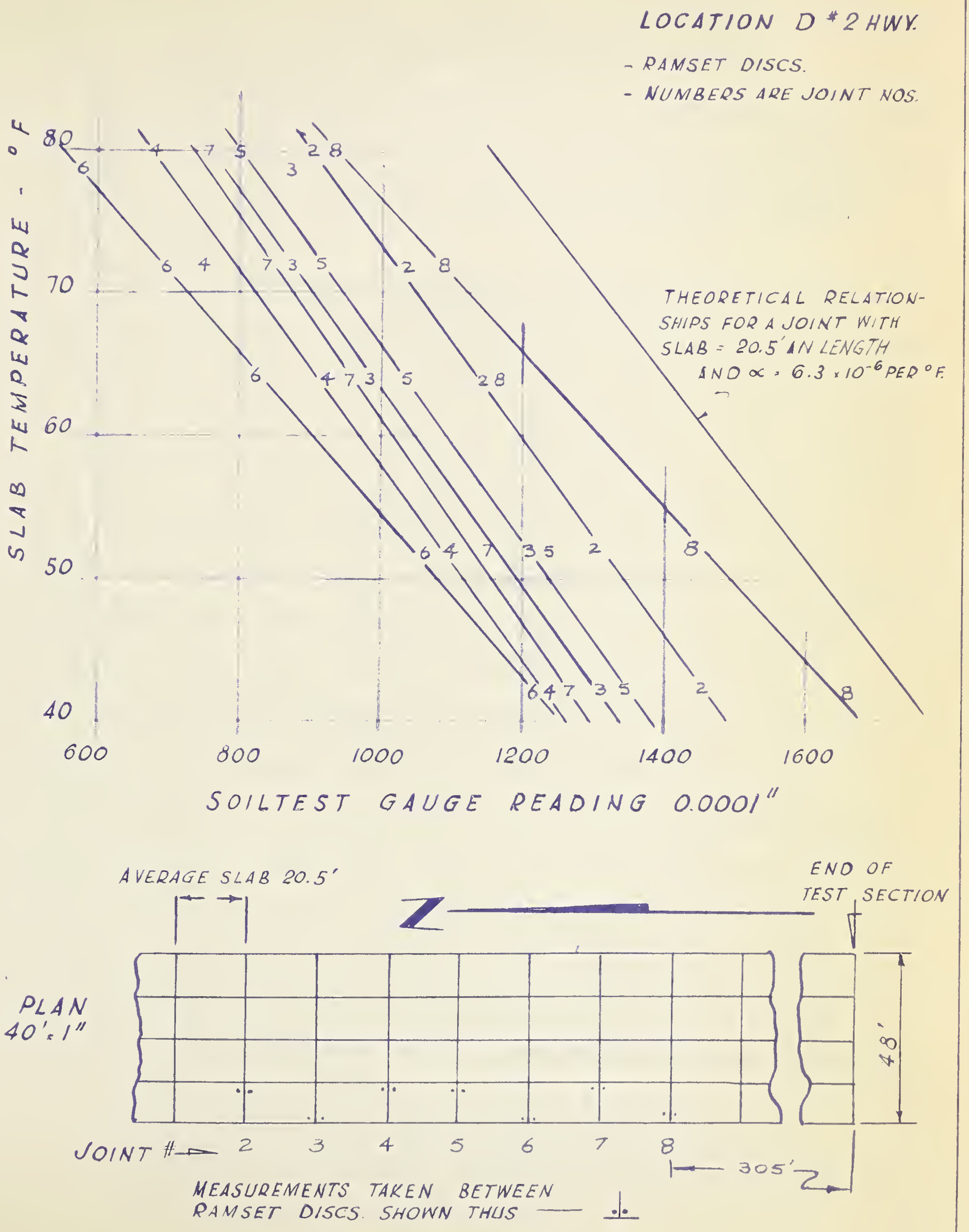


PLATE 12-C

CHAPTER XIII CRACKING IN THE CONTINUOUS (TYPE B) SLABS

Preamble

Research work on continuous pavements was initiated when continuous slabs of various lengths with varying percentages of steel, were laid in 1938, near Stilesville, Indiana.

From observations and measurements of these slabs and their naturally formed cracks, a picture of the behaviour of continuous pavements has been built up, and many fundamental empirical relationships derived.

Several additional experimental continuous pavements have been constructed since the original in Indiana, and these projects have stimulated considerable discussion. Much literature has been published but as yet no theory is universally accepted for design. On some of the newer projects, steel strains have been measured by electrical resistance S.R.4 strain gauges fixed to the reinforcing bars at induced cracks. These gauges have shown that steel strains can depend to a considerable extent upon the time of year that construction takes place, and also that the strains in the bars at any particular crack can vary considerably from bar to bar across the crack.

A laboratory investigation of continuous pavements* concluded that an 8 inch slab, with 0.53 per cent steel 1 inch below centre was better for controlling crack width than 0.77 per cent steel at mid depth, the steel stress with the 0.53 per cent steel being only slightly greater. However, the cracks which occurred in this investigation were minimum in

* M.Gutzwiller and J.Waling "Laboratory Study of Pavements Continuously Reinforced with Deformed Bars" A.C.I. Journal, September 1959.

width at the reinforcement, widening towards surface and base, whereas field cores on actual pavements most often show greatest width at the surface, diminishing regularly towards the base.

Behaviour of the Continuous Slab as suggested by the Literature References

This discussion refers to the fully restrained central portion of a continuous slab. The criterion for maintaining continuity is that the potential strength of the steel must exceed the resultant force from concrete contraction with subgrade friction. A crack will therefore form before the steel itself fails. After several years of temperature fluctuations, equilibrium is reached and the crack frequency remains stable. Thus, if all other factors are equal, there is a well-defined relationship between crack frequency and longitudinal percentage of steel. As would be expected, the crack spacing is least for the heaviest steel, which results in a narrow crack width and low steel stress.

The considerations presented above show that the steel strain is much dependent upon the concrete strength and the manner in which cracking develops. Schiffman, Taylor and Eney * state that "early cracking is primarily the result of shrinkage in the concrete..." They further state that these early cracks continue to open slowly without direct transfer of strain to the reinforcement, probably because shrinkage takes place before bond is fully developed. On their project they found that

* R.Schiffman, I.Taylor and W.Eney "Observations on the Behaviour of Continuously Reinforced Pavements" H.R.B. 1958, unpublished.

shrinkage cracks formed even after rising temperature forced the longitudinal steel into compression.

Curing conditions are likely to have an important bearing both on concrete strength and crack spacing. In Texas, Shelby and McCullough* found relationships among three variables, namely crack spacing at two hundred days, seven day flexural strength and maximum temperature during the first eight hours after pouring (only for concrete poured in the morning). The slab and reinforcement details were the same in each case. The following statements are extracted from their report:-

1. "....it may be concluded that concrete strength has a definite effect upon crack spacing."

   Excessive concrete strengths will result in crack widths that cannot be tolerated if the continuity in a continuous reinforced pavement is to be obtained.

2. "Higher air temperature during curing results in a larger amplitude of shrinkage for an equal period of time. Therefore higher shrinkage stresses during the period when the concrete is relatively weak increases the frequency of cracking".

The function of the steel is to maintain continuity of the slab when cracking occurs, and apart from resisting temperature stresses the steel must transfer load across the cracks. Steel stresses due to traffic loadings will also be very much dependent

* M.Shelby and B.McCullough "Experience in Texas with Continuously Reinforced Concrete Pavements". H.R.B. 1959, unpublished.

upon the crack widths, and thus upon the crack spacing.

The thickness of the slab has not yet been mentioned. In common with all concrete slabs, its adequacy most depends upon traffic loading. However, if the slab is truly continuous no free corners will exist, and a reduction in depth as compared with a conventional slab, is possible.

Thus, the most important single consideration for continuous slabs is the allowable crack width. If crack widths can be kept below a certain limit, load transfer will be mainly by aggregate interlock, steel will not suffer excessive load transfer stresses, and slab thickness can be kept to a minimum. A narrow crack also deteriorates at a much slower rate than a wide crack and retards the entry of water to the base. The maximum crack width for satisfactory transfer of load by aggregate interlock has been set by some authorities at 0.02 inches.*

<u>Summary of Factors Affecting Crack Width</u>

Using the information given above, the factors affecting crack width are summarized below, assuming that crack width is proportional to crack spacing:

1. Increase quantity of longitudinal steel - decrease crack width.
2. Increase concrete strength - increase crack width.
3. Factors tending to increase the rate of initial shrinkage - decrease crack width.
4. Very high air temperatures - increase in crack width (due to creep).

* A.C.I. Committee 325 "Recommended Practice for the Design of Concrete Pavements" Paper #53-39. Page 732 A.C.I. Journal, 1957.

5. Low air temperatures - increase in crack width (due to thermal contraction)

On any particular project, the first three factors depend upon the materials used and the time of year that construction takes place. This governs the temperature susceptibility of the cracks due to thermal changes in the concrete, i.e. factor 5. Climatic conditions would govern factor 4. It should be noted that any decrease in crack spacing, particularly at an early age, when concrete strength is low, would result in a reduction in crack width and steel strain.

Crack Width Data from Illinois *

A qualitative appreciation of continuous pavement behaviour has been presented. An attempt will now be made to accommodate actual figures for crack width, from a pavement with the cracking fully developed. Consider plate 13-A which presents surface crack width data from Illinois at a temperature of 106°F at an age of ten years.

This data shows that for economical percentages of steel, i.e. 0.3 to 0.7 per cent, there is a definite relationship between the crack width and the crack spacing. But the number of cracks per 100 feet is 100 feet/crack spacing, in feet, and if this is multiplied by the average crack width then the value for total crack width per 100 feet is obtained.

The results of this Illinois data show that the surface total crack width per 100 feet is 0.275 inches at 106°F, and this figure appears to be independent of the thickness of slab

* J.D.Lindsay "A Ten-Year Report on the Illinois Continuously-Reinforced Pavement" Figures 4 and 5, H.R.B. Bulletin 214, 1959.

and quantity of reinforcement, and probably depends upon the type of concrete and the conditions under which it was cured.

The total crack width within the slab where the aggregate interlock takes place is probably much less than 0.275 inches. Lindsay felt the crack widths within the slab on the Illinois project were about half the measured widths. Thus, the value of 0.275 inches represents a width of 0.137 inches within the slab where aggregate interlock and load transfer take place.

If the greatest stress condition in the steel is assumed to occur at 30°F, i.e. at the time of the spring break-up when the subgrade is weakest, and if the coefficient of linear expansion for the reinforced slab is 6.0×10^{-6} inches/inch/°F, then the crack spacing necessary to keep the average crack width at 0.02 inches at 30°F will be approximately

$$\frac{0.137'' + 6.0 \times 10^{-6} \times 12 \times 100 \times 76}{0.02} = 34 \text{ cracks per 100 feet}$$

i.e. one crack every 3 feet. (This is assuming that the total crack width per 100 feet can be expressed as:

$$\text{width at } T^\circ F = 0.137 + \alpha (106 - T^\circ) \times 100 \times 12)$$

A more accurate estimate could be made if a plot of "average crack width within the depth of the slab per 100 feet" versus temperature, were available. Such data from the type B pavements would be useful.

Wolley* suggested that 0.3 per cent steel is sufficient to resist temperature stresses alone. Thus, 0.3 per cent steel in a slab with a uniform 3 foot crack spacing, if such a pavement could be constructed, should produce minimum stresses in the steel. Although such a design may be worth of further consideration, a crack spacing of 3 feet could partly destroy the load distribution effect of the slab.

* "Design of Continuously Reinforced Concrete Pavement" H.R.B. Bulletin 181, 1958.

Thus, if the data presented above is accepted, the indications are that traffic stresses in the steel cannot be completely eliminated by crack control.

Crack Observations on the Type B Pavements

The observations and measurements on the type B pavements have revealed certain weaknesses in allowing the cracks to self form. These weaknesses are the result of uneven width and uneven crack spacing.

1. A typical plot of crack width versus temperature at an induced crack is compared on plate 13-B with a naturally formed crack nearby. A plan is given on plate 13-C. The induced crack which is at an SR4 strain gauge location might be typical of construction joints and shrinkage cracks occurring naturally at an early age. It can be seen that the induced crack is many times more susceptible to temperature changes than the later developed natural crack. (All four natural cracks examined were about the same width.) Thus, the margin against failure at the induced crack is probably much lower than the nearby natural cracks. Thus, the margin against failure of the construction joints and some of the earliest shrinkage cracks is probably much lower than the majority of the other cracks.

It is possible that if all cracks had occurred, or had been formed, simultaneously, then each would have had the same susceptibility to temperature changes.

2. The cracking pattern is highly irregular and in several places the crack interval is less than two feet. (See plate 13-D). It is possible that the continuous pavement with such a frequent crack spacing will behave as a semi-rigid road structure and the beam action of the slab partially lost.

From the points mentioned above, it seems that some artificial control should be exercised, if possible, over the cracking interval. A localized stress concentration in the form of a thin metal strip or a wire, within the slab, might produce the requisite cracks. It was noticed that cracks very frequently passed through the elevation bolts near the edge of the slab, and it is felt that this is due to stresses induced in the surrounding concrete by the hole that accommodates the bolt.

If the requisite number of cracks could be induced at the desired spacing when the concrete was just a few hours old, there would be more likelihood of uniform strain conditions within the steel at each crack. To achieve this end, the action of stress inducers in a laboratory study could be tried.

The principal question of doubt that remains in the hypothesis that "cracks formed at the same time would have the same width in subsequent years" is the yielding of the steel. It is possible that some cracks would yield before others, thus leading again to uneven crack widths. (Crack width due to yielding can amount to about 0.04 inches as shown on plate 13-B).

Cracking Record

A record has been made periodically of the cracks per 100 feet and during the survey in July, 1959, the cracks were painted at the south edge of the slab. The cracking record is given on plate 13-E.

Within the first portion paved, between stations 375 and 341, the initial rate of cracking was high and almost half of the present cracks formed before the age of two weeks. It

appears that the ultimate crack spacing will average slightly more than the 16 per 100 feet for 0.7 to 0.8 per cent steel in the central portions in Indiana and Illinois. (The continuous pavements in Indiana and Illinois utilised no base, which may have some bearing on this matter because of the differences in frictional characteristics between gravel base and clay subgrade.)

After two days of "no pour" at station 341, a marked difference in behaviour was noticed. Significant cracking did not start to develop until the slab was older than two weeks, and the development of cracking was much more irregular. This irregularity in development of cracking could in part be due to the following:

1. The four day "no pour" enabled the joint at station 324 to behave like a partly free end.
2. The close spacing of the strain gauge locations V, VI and VII, with their artificially induced planes of weakness, could upset the natural behaviour.

Since a large proportion of the cracks prior to station 341 occurred at an early age, the performance of this section should be better than the remainder of the type B pavement. The width of a crack necessary to absorb slab expansion and contraction depends upon the crack spacing. Thus, the earlier cracks, after station 341, would tend to be wider than the earlier cracks before 341, and therefore more likely to fail.

For these particular slabs on #1 Highway there appears to be no immediately obvious correlation between the flexural strength,* air temperature during pouring, and crack spacing.

Final conclusions concerning the crack spacing cannot be

* Concrete Strength data was taken from Appendix B.

drawn for several years until the pattern is fully developed.

Longitudinal Crack

A longitudinal crack, as shown in plate 13-F, was observed in the summer of 1959.

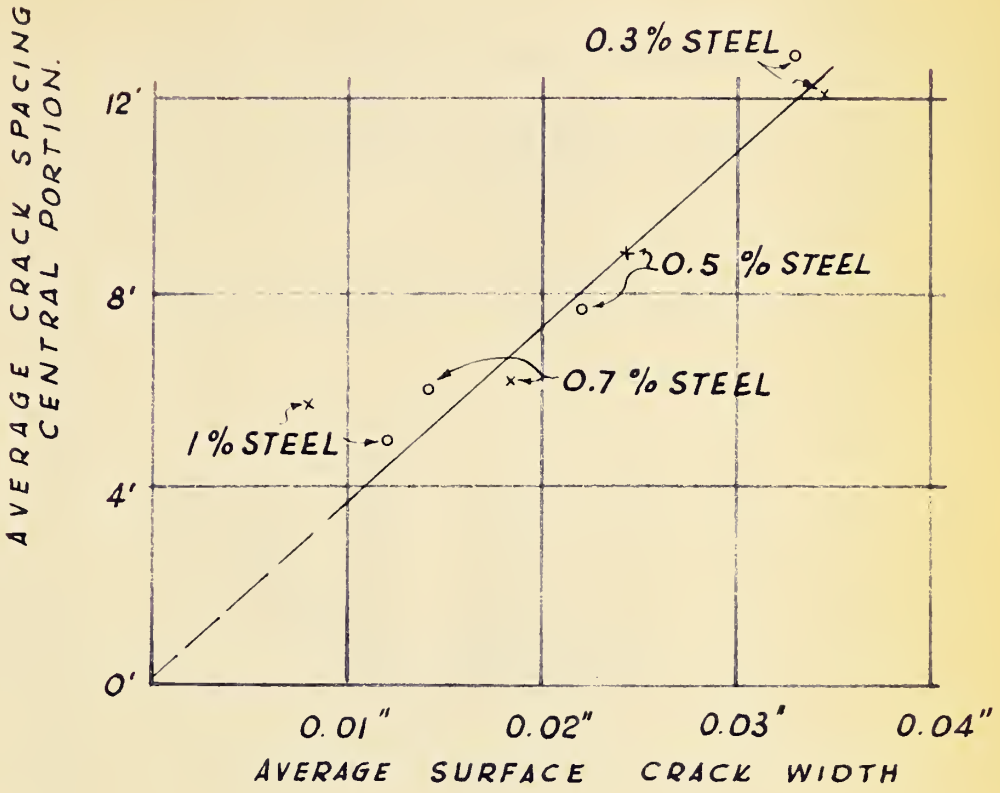
It is felt that it was induced by the reinforcing bars, because in the middle it "jogged" a distance equal to the bar spacing. The reinforcing bars might have been too close to the surface, which could be checked by a core.

In the summer, this crack appears to be in good condition, but the most recent survey (January 1960) revealed some deterioration.

Appearance of Cracks

The majority of cracks appear in good condition as in the photographs on plate 13-G. This photograph illustrates, however, a significant weakness of continuous pavement design. This is that the "islands" that form during cracking eventually deteriorate into spalls, and finally cause rough riding surface. This weakness, however, can be readily overcome by a thin hot plant mix surface course, whereas with jointed pavements a bituminous resurfacing is seldom fully satisfactory due to the excessive movement at the joints.

CONTINUOUS PAVEMENT IN ILLINOIS



LEGEND.

o – 8" SLAB
x – 7" SLAB.

AGE: 10 YRS. TEMP. 106° F.

REFERENCE: J. D. LINDSAY
H. R. B. BULLETIN 214

PLATE 13-A

CRACK WIDTH VARIATIONS IN A CONTINUOUS CONCRETE PAVEMENT

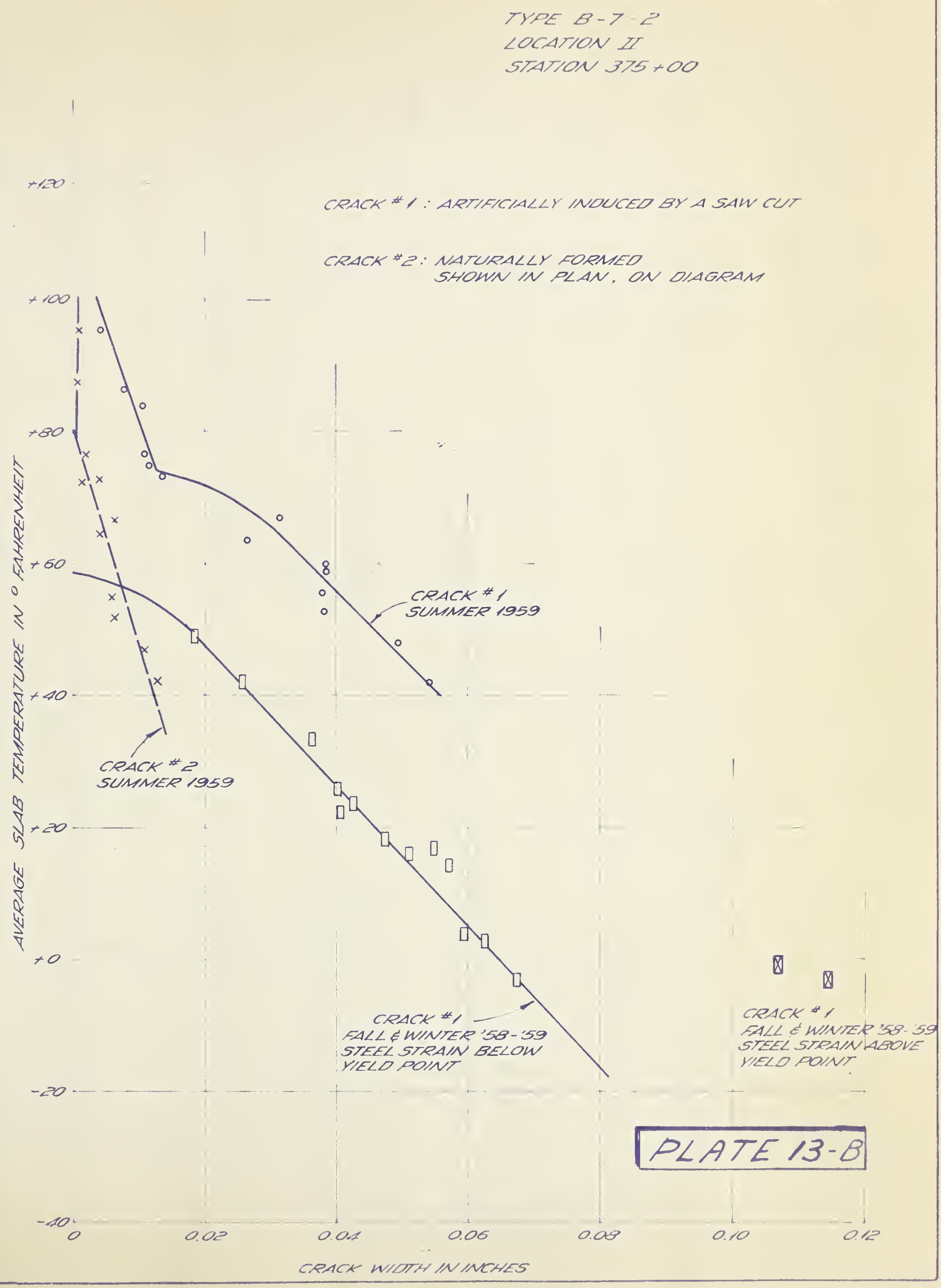
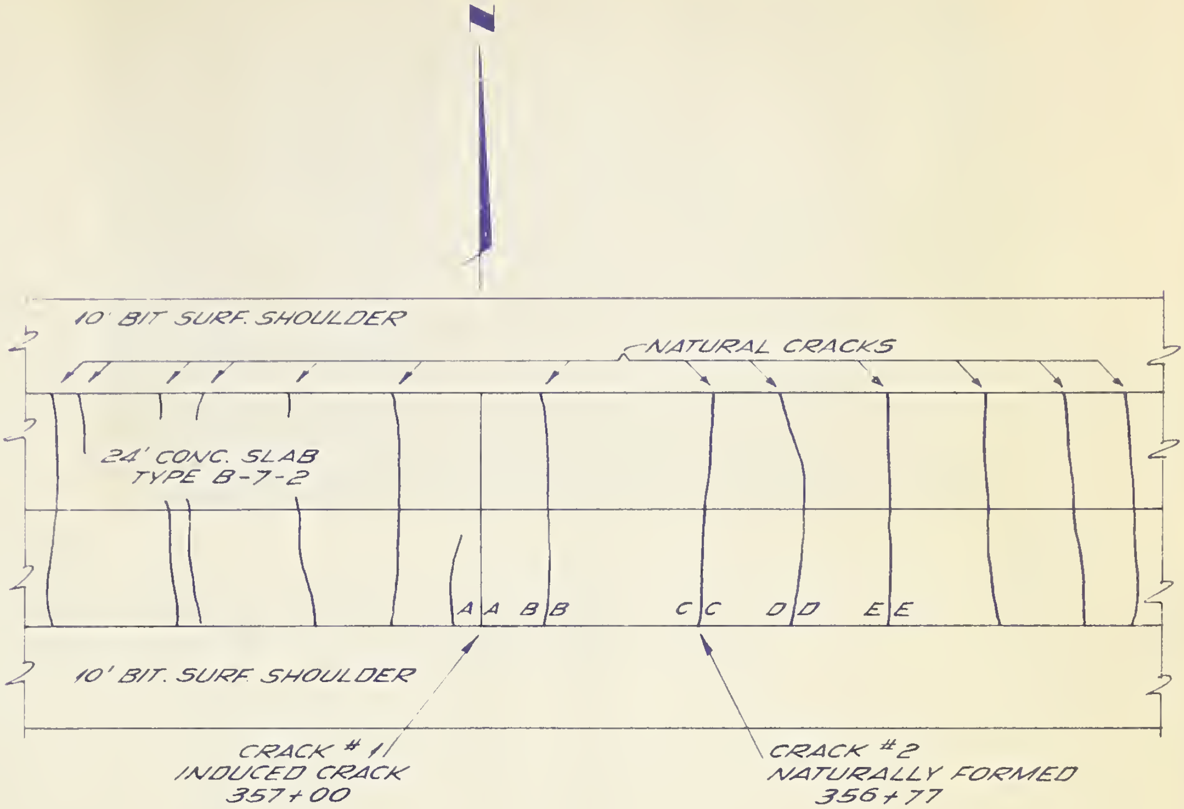


PLATE 13-B

CRACK MEASURING DEVICES AT STRAIN GAUGE - LOC. II



SCALE 1" = 20'

CRACK PINS MEASURED AT AA

RAMSET DISCS MEASURED AT BB, CC, DD, EE

PLATE 13-C

EXAMPLES OF CRACK SPACING ON THE CONTINUOUS PAVEMENTS

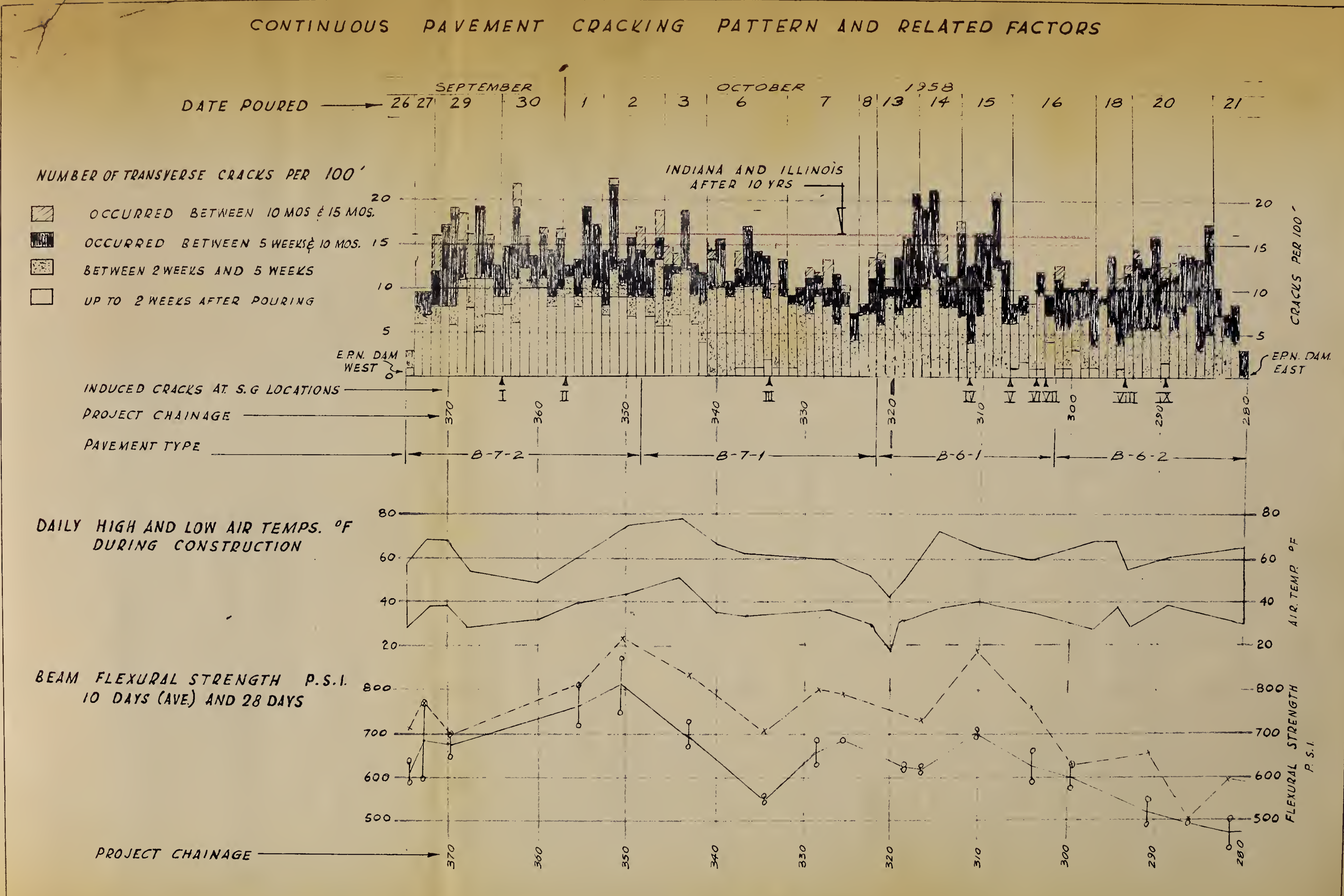
FIG 1

THE PHOTOGRAPH ILLUSTRATES IRREGULARITY OF CRACK SPACING IN THE 15' LENGTH OF PAVEMENT DESIGNATED BY THE ARROW, THERE ARE SEVEN SEGMENTS i.e CRACK SPACING IS 2.1'

FIG. 2

THE PHOTOGRAPH ILLUSTRATES THE EFFECT OF A CONSTRUCTION JOINT UPON THE CRACK SPACING.
CONSTRUCTION PROCEEDED FROM THE BACK OF THE PHOTO AND THERE WAS A TIME INTERVAL OF THREE DAYS AT THE JOINT

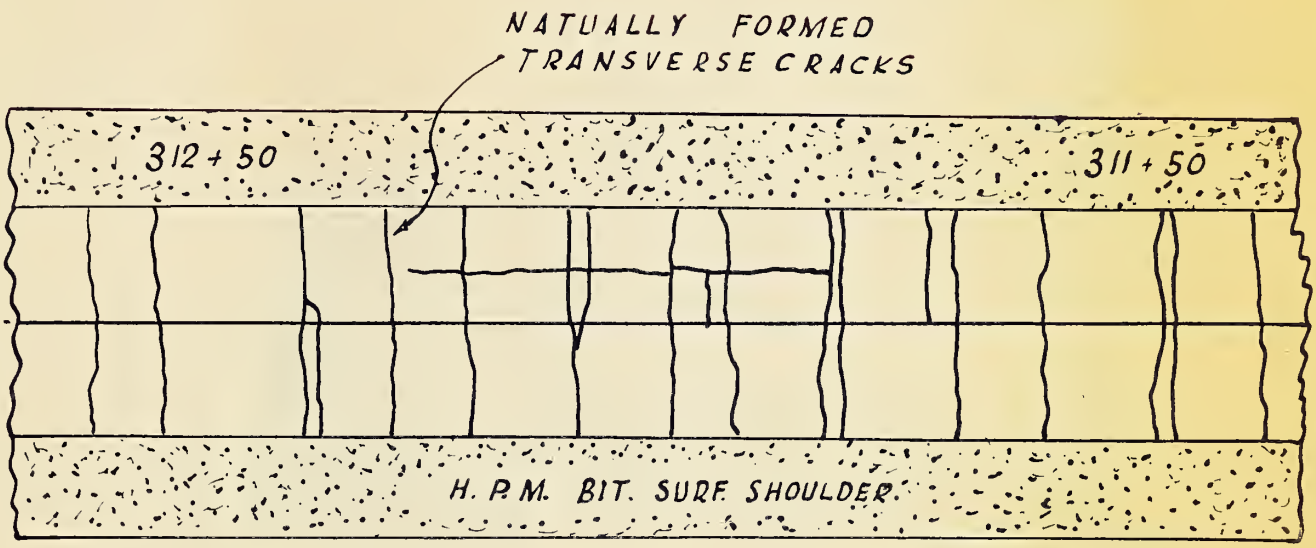
PLATE 13-D



CONTINUOUS PAVEMENT CRACKING PATTERN AND RELATED FACTORS
DATE POURED
SEPTEMBER
OCTOBER
1958
26 27 29 30 1 2 3 6 7 8 13 14 15 16 18 20 21
NUMBER OF TRANSVERSE CRACKS PER 100'
OCCURRED BETWEEN 10 MOS & 15 MOS.
OCCURRED BETWEEN 5 WEEKS & 10 MOS.
BETWEEN 2 WEEKS AND 5 WEEKS
UP TO 2 WEEKS AFTER POURING
INDIANA AND ILLINOIS AFTER 10 YRS
E.R.N. DAM WEST
E.R.N. DAM. EAST
CRACKS PER 100'
INDUCED CRACKS AT S.G LOCATIONS
I II III IV V VI VII VIII IX
PROJECT CHAINAGE
370 360 350 340 330 320 310 300 290 280
PAVEMENT TYPE
B-7-2
B-7-1
B-6-1
B-6-2
DAILY HIGH AND LOW AIR TEMPS. °F DURING CONSTRUCTION
AIR TEMP. °F
BEAM FLEXURAL STRENGTH P.S.I. 10 DAYS (AVE.) AND 28 DAYS
FLEXURAL STRENGTH P.S.I.
PROJECT CHAINAGE

PLATE 13-E

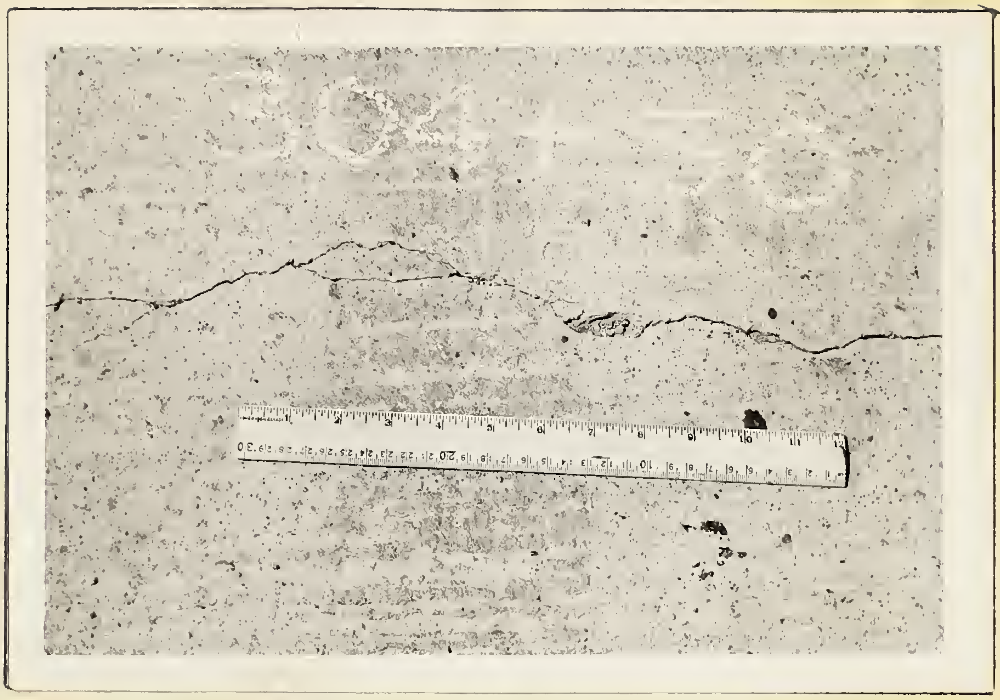
LONGITUDINAL CRACK IN TYPE B-6-1



AGE - 10 MONTHS
WIDTH - 1/16"
SURVEY AUG 1959
BY W. BABOWAL

SCALE 20' = 1"

PLATE 13-F



"ISLANDS" DEVELOPING INTO SPALLS
– AGE 10 MONTHS

PLATE 13-G

CHAPTER XIV EXPANSION DAM MEASUREMENTS

Construction drawings of the expansion dams are given in Appendix A.

West Expansion Dam

The west expansion dam is being closed up because the slab immediately to the west is moving downhill. Only half an inch of space existed at 92°F in July, 1959.

The expansion joint between this uphill slab and the type C pavement is opening up, as shown in plate 8-B, figure 2. This is tending to reduce the available space at the expansion dam.

East Expansion Dam

In the spring of 1959 a difference in leval of about 3/8 inch between the two sets of tongues on the east expansion dam was noticed.

Levels were taken to check for any settlement of the adjacent slabs, and although this location is on a fairly high fill, no measurable settlement was recorded. It is possible that the base slab has tilted slightly, either because of settlement, or because the continuous slab is sticking to it as it moves.

Measurements taken at this dam and presented on plate 14, show a closure of about 2 inches during the first year after pouring. An examination should be made to see if the joints in the adjacent conventional slabs are opening up.

Comparison with Data from other Projects

If the joints adjacent to the east expansion dam in the type A-6 pavement have not opened and absorbed space at the expansion dam, then the overall length change of the type B slabs can be estimated at 4 inches expansion during one year.

Cashell and Benham* found that the average increase in length of sections 1070 feet long, or longer, was about 3/4 inch after one year, increasing steadily to be about $2\frac{1}{2}$ inches after ten years. The amount of space required to absorb expansion at 92°F after ten years was 3.8 inches.

After ten years, Lindsay+ concluded that the Illinois continuously reinforced pavements were about 4 inches longer than their original lengths.

Thus, the rate of closure at the east expansion dam in the type B pavements on #1 Highway seems to be excessive compared with these other projects.

* "Experiments with Continuously Reinforced Concrete Pavements" Proceedings, H:R:B 1949, figure 12, page 56.

\+ "Ten Year Report on the Illinois Continuously Reinforced Pavements" H.R.B. Bulletin 214, page 26, 1959.

MEASUREMENTS AT THE EAST EXPANSION DAM

STATION 280+03

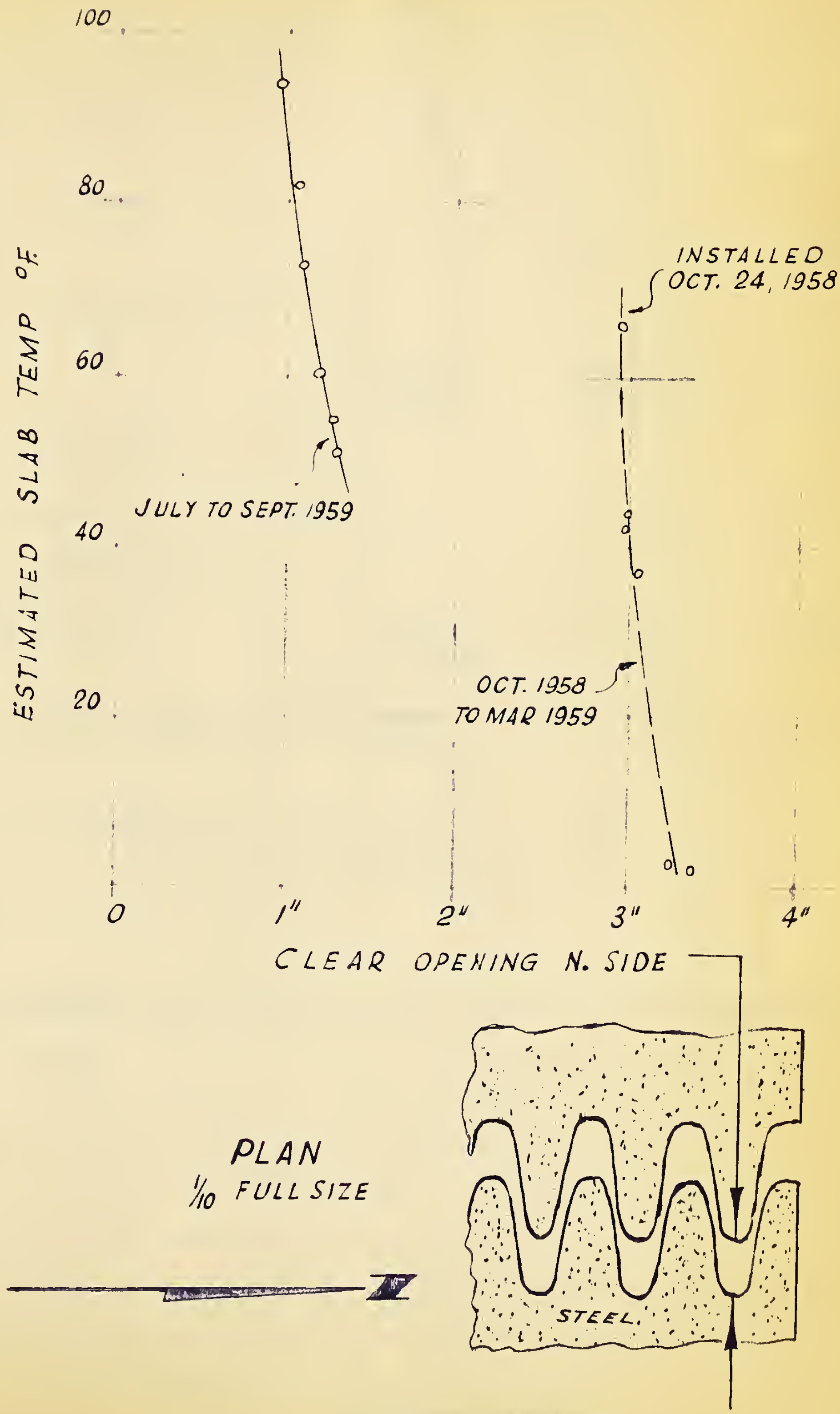


PLATE 14-A

CHAPTER XV SUMMARY OF DATA FROM JOINT WIDTH MEASUREMENTS

Table 15-1

Location	A-8-6	C-14	A-6-18
Station	161	411	279
Slab length - feet	29.8 S 30.1 N	20	30
Slab depth - inches	8	8	6
Reinforced	Yes	No	Yes
Mix design (see Table 3-2)	4	1	2
Date poured	May 28, 1959	Sept.18, 1958	Oct.25, 1958
Pouring temperature ^{o}F	55	-	-
To - early age ^{o}F	63	63	45
To - max.summer 1959 ^{o}F	74	81	-
α ins/in/^{o}F x 10^{-6}	5.4(S) 5.7(N)	6.6(S)	5.9(S)
Curing shrinkage ins/in x 10^{-6}	48(N) 39(S)	-	-
Max.Creep shrinkage ins/in x 10^{-6}	60(N) 47(S)	108	

These measurements were taken between the date of pouring and September, 1959.

CHAPTER XVI SUMMARY OF OBSERVATIONS AND CONCLUSIONS

These conclusions are drawn from observations and measurements carried out between September 1958 and January 1960.

The pavements examined were plain unreinforced slabs 20 feet x 12 feet x 8 inches with no dowels, called type D; plain unreinforced slabs 20 feet x 12 feet x 8 inches with doweled joints, called type C; reinforced slabs 30 feet x 12 feet x 6 inches, 7 inches and 8 inches, called type A-6, A-7 and A-8 respectively, and continuously reinforced slabs of four different designs, called type B.

The type D was constructed in June 1955 on #2 Highway just north of Calgary. The types C, B and A-6 were constructed in September, 1958, and the types A-7 and A-8 in May 1959, all on #1 Highway, west of Calgary.

General Remarks on Cracking

Observations of cracks in the plain concrete slabs revealed that all cracks must be considered a definite structural defect because they tend to widen, spall and fault. The cracks in the type A pavements are not yet structural failures.

Types of Cracking in the Jointed Pavements

The majority of the significant cracks are transverse cracks. There is no evidence of restraint cracking, corner cracking, longitudinal cracking or cracks due to excessive settlement. Numerous shrinkage cracks have occurred, but these have not led to any structural failures. Also, on type C, cracks and spalls have occurred near sawn joints.

Transverse cracks Prior to Opening the Road to Traffic

In the plain unreinforced slabs, many of the serious

transverse cracks occurred before the road had been opened to traffic. In type C, these cracks amounted to 10 in a present total of 17, and on type D, 22 in a present total of 26. In type C it is felt that these early formed cracks are the result of stiff dowels.

With the type D many of these earliest cracks are sympathetic with joints sawn in adjacent slabs. They occurred during the initial shrinkage period by reaction to thermal contraction within the mature slabs on the adjacent green slabs. Otherwise they occurred because the joints were not sawn in time.

Cracks and Spalls during Sawing

Other cracks on the type C occurred during sawing, probably because the concrete was sawn too green.

Performance Based on Rate of Cracking

The earliest cracks on types C and D are not typical of concrete pavements elsewhere and could probably be eliminated by slight changes in design and construction procedures. If these earliest cracks are neglected and performance is based on rate of cracking, a reflection of the response of the pavement to repetitions of traffic loading and warping stresses should be obtained.

The rate of cracking on type C is 0.43 feet per 12 feet x 20 feet slab per annum, and is far greater than the type D, which is cracking at about 0.02 feet per 12 feet x 20 feet slab per annum. Thus the performance of type D is satisfactory, and that of type C not nearly so good in comparison. This is anomolous because types C and D are almost identical in construction except

that the type C has dowels, which should actually improve the performance. (Complete comparison cannot be made because the traffic loading is different in each case.) But it is felt that the effect of stiff dowels may still be causing the cracking on type C.

Cracks in the Reinforced Slabs Except Location A-8-4

The number of cracks in the type A slabs, except for location A-8-4, is not excessive and the performance can be considered as quite satisfactory. To date, no advantage of the heavier type A-8, over type A-6, has shown up.

Cracks at Location A-8-4

Six of the eight joints in a length of nine consecutive 30 foot slabs did not open. This resulted in frequent uncontrolled cracking. Actually, eighteen uncontrolled cracks occurred in these nine slabs, but as yet are showing no signs of distress.

Shrinkage Cracks

On the type C pavements there are many groups of shrinkage cracks, and these usually occurred when the Contractor had difficulties in getting started in the morning.

In future, careful control should be exercised to see that brooming is carried out as soon as possible after pouring, and curing carried out immediately after brooming. The techniques developed by the Concrete Engineer on pavement type A-8 seem to work admirably. Shrinkage cracks also occurred in small areas on type B and in one area on type A-7.

Joint Faulting

There is no observable faulting at any of the doweled joints,

but some slight faulting is occurring at the undoweled joints on type D.

Joint Spalling

Spalling at the joints is slight at the doweled joints, but is worse on the type D slabs which are three or four years older. In general, it can be stated that joint spalling is not yet serious.

Joint Sealer

The joint sealer tends to be spread by the traffic and collects small stones. This is likely to cause considerable uneveness in the future. Care should be taken not to overfill joints during maintenance.

Pumping

There is no evidence of pumping from the joints, but on #2 Highway it was noticed that some of the joints had been fairly recently sealed. This would considerably reduce the effect of pumping for a year or two if any were incipient.

Water Oozing from Joints

This was noticed from the joint between the shoulder and slab at about twenty-three locations on the types A, B and C pavements. By far the worst locations were on type A-7 and A-8 near the bottom of a 5 per cent grade about 2,000 feet long. Water was seen to ooze from a transverse joint and the centre line joint, at this location also.

It is considered that the majority of this water is rain-water and enters through the joint between shoulder and slab. If this water leads to serious defects, some reconsideration of

the base course specification might be necessary.

Settlement of the Shoulder at the Edge of the Slab

The shoulder rapidly settles down from the slab and causes a step of about $\frac{1}{2}$ inch to 1 inch, which is a minor traffic hazard. In order to alleviate this settlement, steady maintenance has been necessary. It might be possible to lessen this settlement in the future by strengthening the shoulder adjacent to the slab.

Surface Condition of the Pavement

The surface finish on the type A pavements is superior to the type C, with the type B somewhere between. Ideally, the finished surface of a slab should present a crust of mortar which is broomed at the time of initial set. This presents a smooth riding surface with good tire grip. On type C, when high spots were removed by hand, it was noticed that the mortar layer seemed to erode fairly rapidly to expose the aggregate and leave a rough surface.

The type D is three years older than type C but its surface shows no marked deterioration. Holes left by clay lumps are more frequent on the type D than on types A, B or C.

Riding Quality

Riding quality was measured by a HiLo Roughometer manufactured by "Soiltest" of Chicago. It consisted of a perambulator with two wheels on a longitudinal span of sixteen feet. At mid span a recording wheel acting on the surface of the slab was free to move vertically, operating a pointer on a dial fixed to the span. The instrument was pushed at slow walking speed and the readings were recorded manually. Ellert

performed an analysis which gave an "irregularity index" in inches/mile.

Providing the same results can be reproduced, under the same conditions, the method seems to be satisfactory for comparing the riding quality and deterioration of riding quality of different types of construction. However, the results cannot be directly compared with the results from elsewhere using different methods; perhaps a correlation should be arranged.

Readings were taken on the type C, type B and type A-6 pavements before much traffic had used the road. Thus, the survey reflected the effect of joints and finishing techniques. With uniform finishing the type B would be expected to give the least roughness and the type C (because of the most frequent joints) the most roughness. However, contrary to this expectation, the type A-6, the last constructed, was smoother than the type B. It seems that the greatest majority of rough riding derives from the operation of finishing the concrete.

Joint Crack Behaviour

In the doweled joints the initial shrinkage first caused cracking on an average spacing of 90 feet, with a range of between 60 feet and 150 feet. These joints have been called wide joints, and have high susceptibility to temperature changes. Subsequent shrinkage and temperature movement caused cracking at most of the remaining joints, and these tend to have considerably less susceptibility to temperature changes than the first formed joints. These latter joints have been called narrow joints.

On the completion of cracking, it was expected that the joints would equalize in width. However, this did not occur and the susceptibility of the wide joints stayed many times greater than the narrow joints. The ratio of temperature susceptibility for a wide joint to the "average" joint is between 1.3 and 3.0, using the measurements shown in the results. But, for the narrow joints, this ratio is only between 0.7 and 0.1

Temperature Susceptibility of the Joints on the Type D pavement

From a sample location of seven consecutive joints, it was discovered that each joint has almost the same susceptibility, which is about equal to the coefficient of linear expansion for concrete slabs.

Thus, it can be concluded that the system of wide and narrow joints occurring in the doweled pavements, although initiated by shrinkage effects, is maintained by variations in dowel stiffness from joint to joint.

Improvements in Dowel Setting

If the type of design used on #1 Highway is tried again, the performance could be improved by carefully controlling the placing of the dowels to give each an exact alignment with the surface and side of the slab.

Suggested Trial Designs

The margin against failure is much greater at the wide joints than at the narrow joints. Hence the narrow joints in reinforced slabs could be perhaps eliminated. A trial could be made using reinforced slabs 100 feet in length, with high load transfer, low resistance, corrosion free dowels in the formed

joints. This would follow recent design trends in the U.S.A. and Ontario.

With unreinforced slabs, a high load transfer low resistance, corrosion free doweled joint should be formed at 100 feet intervals; with the ordinary doweled joints sawn at 20 feet intervals betwee. These designs might blend in better with the manner in which the doweled joints seem to behave.

Creep

Hot weather caused the joints to close completely, and induced compressive stresses within the concrete. This resulted in concrete creep, but this creep was found to be recoverable, at least in part, during subsequent cooler weather. This type of behaviour could be called "weather creep". The creep strain in the type C (unreinforced) pavements, was twice as great as on the type A-8 reinforced slabs.

The Effects of Creep

This creep tended to even up the differences in joint widths, probably by loosening the stiffer dowels in the narrow joints. Thus, the temperature susceptibility ratio of a wide joint to average joint was 3.0 in the fall soon after pouring, and 1.9 in the fall of the following year.

Also, creep may be one reason why blow-ups were not observed.

Values Obtained for Linear Expansion, Shrinkage and Creep

Joint width measurements at different temperatures at different times of the year over a series of adjacent joints, enabled the determination of field values for the coefficients

given above. The results are given in detail in Table 15-1. Values obtained for the coefficient of linear expansion were between 5.4 and 6.6 x 10^{-6} inches/inch/^{o}F, for curing shrinkage about 44 x 10^{-6} inches/inch/^{o}F, and for creep about 53 x 10^{-6} inches/inch/^{o}F with reinforced slabs, and 108 x 10^{-6} inches/inch/^{o}F with plain slabs. The value for the curing shrinkage is only about 10 per cent of the values given by two principal references.*

Centre Line Joint on #1 Highway

Although the centre line joint is suspected of having cracked in some places, it appears that this does not readily occur. It could be that the centre line joint does not readily crack due to loading, because the shoulder keeps moisture fairly constant beneath the slab, resulting in uniform support. (Strong shoulders were not common on earlier concrete highways.)

Transverse Steel in the Type A Pavements

The quantity of transverse steel is almost equal to the quantity of longitudinal steel. The possibility of reducing the quantity of transverse steel should be considered.

Cracking Patterns in the Continuous Pavements

Although the cracking pattern is not fully developed, it appears that the final frequency will be slightly more than experienced on pavements of similar design in Indiana and Illinois.

The cracking pattern seems to be affected by the strain gauge locations, when they are close together, and construction joints, if there is a wait of several days.

* D.S.I.R. "Concrete Roads" Table 4.3

Bureau of Reclamation "Concrete Manual" page 41

Possible Approach to Design of Continuous Pavements

It appears that a very important factor governing the performance of continuous pavements is the crack width, since steel stress is closely related to crack width. Crack widths could possible be controlled at an equal minimum by inducing cracks at a desired uniform spacing at an early age. This might lead to reductions in steel stresses and economy in design.

Close Spacing of the Naturally Formed Cracks

On the type B pavements, there are some stretches with eight or so cracks, spaced at about 2 foot intervals. It is possible that a slab with such a close interval of cracking would act as a semi-rigid pavement. Hence, this type of behaviour is undesirable, and is another reason for suggesting some artificial control of crack spacing.

Expansion Dams between the Continuous and the Conventional Slabs

In one year of life, the expansion dam on the west end has closed from about 3 inches to $\frac{1}{2}$ inch, but this is in part due to an adjacent 30 foot slab sliding from its neighbours.

The east expansion dam has closed from 3 inches to 1 inch, but the reason for this is not directly obvious. Also, at this dam, the east side has risen 3/8 inch above the west side, and the reason for this is as yet unknown.

The closure of these dams is considerably more than would have been expected by considering data from Indiana and Illinois.

CHAPTER XVII BRIEF COMPARISON OF THE DIFFERENT TYPES OF CONSTRUCTION BASED ON THE BEHAVIOUR TO DATE

At present, the lightest continuous slab, i.e. 6 inches with 0.72 per cent steel, is performing as satisfactorily as the heavier section which is 7 inches thick with 0.82 per cent steel.

The same applies to the type A-6 in comparison with the types A-7 and A-8. However, type A-6 has an 8 inch base, whereas the types A-7 and A-8 have a 4 inch base.

Experience with uncontrolled cracks on the type D pavement, which is now four years old, seems to indicate that deterioration of uncontrolled cracks is rapid, and that cracks in plain slabs represent a definite structural defect.

Many of the cracks in the plain slabs are not due to inadequacies in design, but more to unfortunate construction techniques and seized doweled joints, and it may be unfair to rate the performance by them.

However, it seems that adding steel provides an excellent factor of safety against unforeseen problems. Thus, as things stand at present, a 6 inch slab with reinforcement and dowels, is preferable to an 8 inch plain slab with or without dowels.

It is suspected that the plain concrete without dowels will perform better than the plain concrete with dowels. Because of the dowel resistance, the uncontrolled cracks in the type C will tend to absorb the thermal movements, and will therefore develop wider than those in the type D.

No rating between the continuous slabs and the conventional slabs is yet possible. This rating will probably hinge upon the behaviour of the doweled joints. These might fail structurally,

or deteriorate sufficiently at the surface to cause overall failure of the pavement from the point of view of poor ridability.

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29. H.Cashell and W.Teske "Continuous Reinforcement in Concrete Pavement" H.R.B. Proceedings, 1955.

30. M.Shelby and B.McCullough "Experience in Texas with continuously Reinforced Concrete Pavement" H.R.B. 1959 - unpublished.

31. W.Wolley "Design of Continuously Reinforced Concrete Pavement" H.R.B. Bulletin 181, 1958.

32. R.Schiffman, I.Taylor and W.Eney "Observations on the Behaviour of Continuously Reinforced Pavements in Pennsylvania". H.E.B. 1958, unpublished.

33. M.Gutzwiller and J.Waling "Lab Investigation of Continuous Pavements" A.C.I. Journal, September 1959.

34. J.D.Lindsay "A Ten-Year Report on the Illinois Continuously-Reinforced Pavement" H.R.B. Bulletin # 214, 1959.

35. W.Van Breemen "Ten-Year Report on Experimental Continuously-Reinforced Concrete Pavements in New Jersey" H.R.B. Bulletin # 214, 1959.

36. B.Tremper "Continuously-Reinforced Concrete Pavement in California after Eight Years Service" H.R.B. Bulletin # 214, 1959.

37. F.C.Witkoski and R.K.Shaffer "Continuously-Reinforced Concrete Pavement in Pennsylvania" H.R.B. Bulletin # 214, 1959.

38. I.J.Taylor and W.J.Eney "First-Year Performance Report on Continuously-Reinforced Concrete Pavements in Pennsylvania" H.R.B. Bulletin # 214, 1959.

39. B.F.Friberg "Frictional Resistance under Concrete Pavements and Restraint Stresses in Long Reinforced Slabs". H.R.B. Proceedings 1954.

40. J.Bereznicki "Direct Measurement of Strain in Concrete Pavement Steel". Unpublished M.Sc. Thesis, U.of A., Dept. of Civil Engineering, Edmonton.

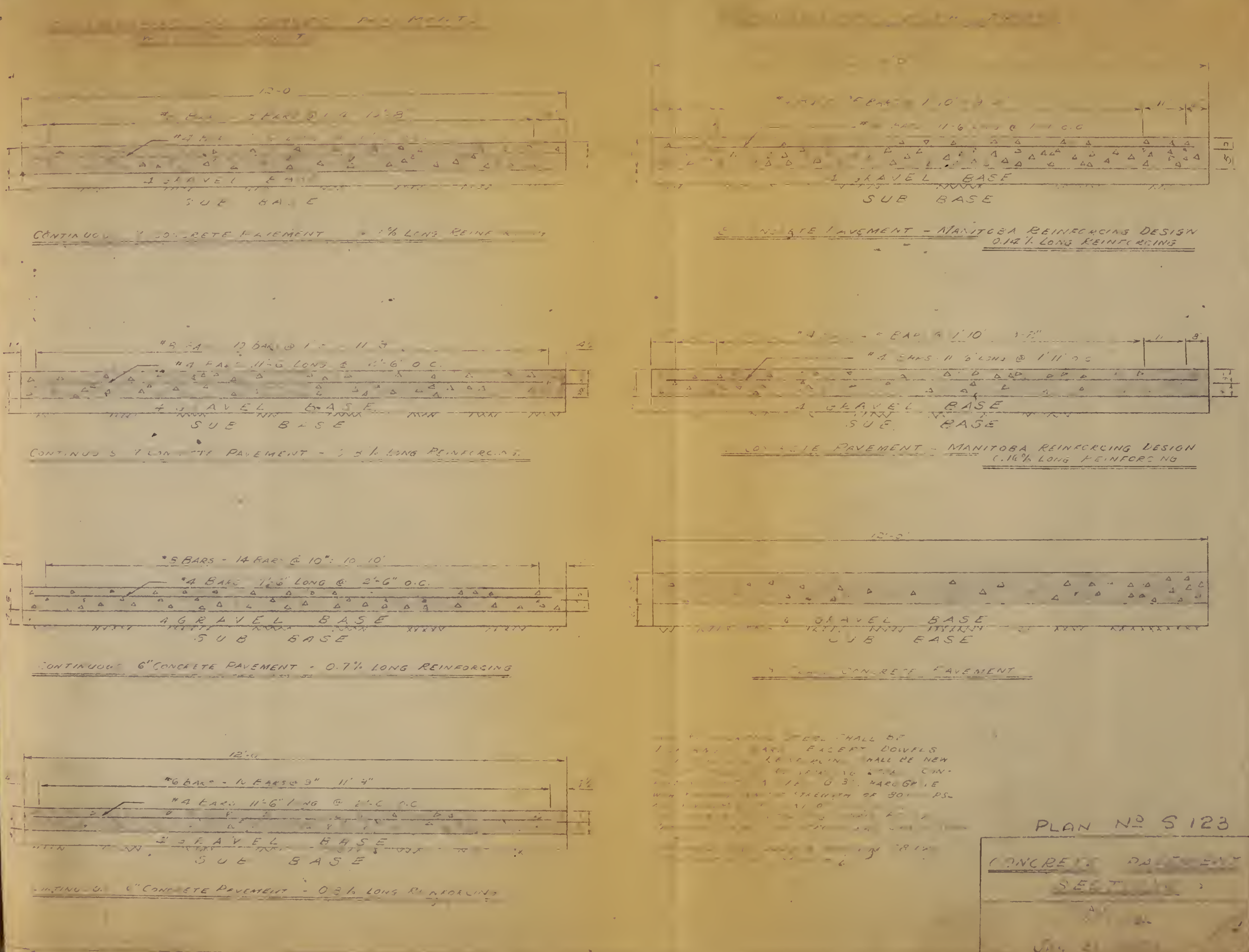
41. Private Communications with "Stelco" of Hamilton, Ontario, 1959.

42. A.C.I. Committee 325 "Design Considerations for Concrete Pavement Reinforcement for Crack Control". Appendix B, A.C.I. Journal, 1956-57.

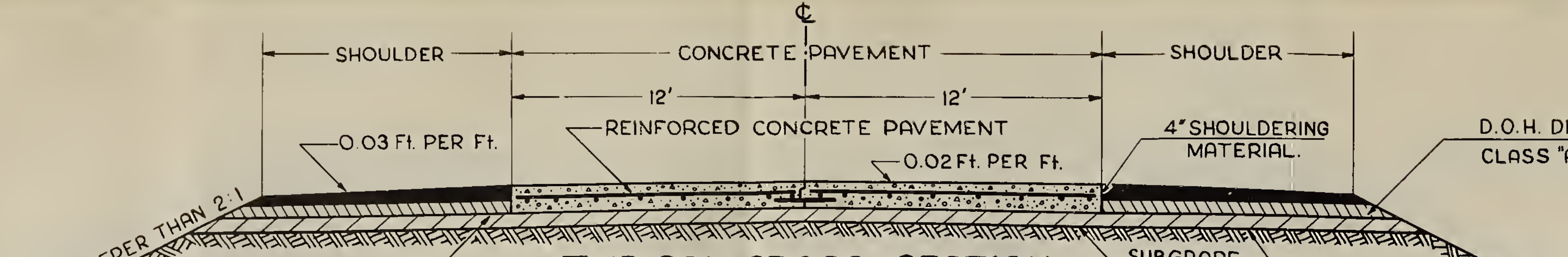
A P P E N D I X   A

ALBERTA
DEPARTMENT OF HIGHWAYS
CONSTRUCTION DRAWINGS
FOR THE PROJECT

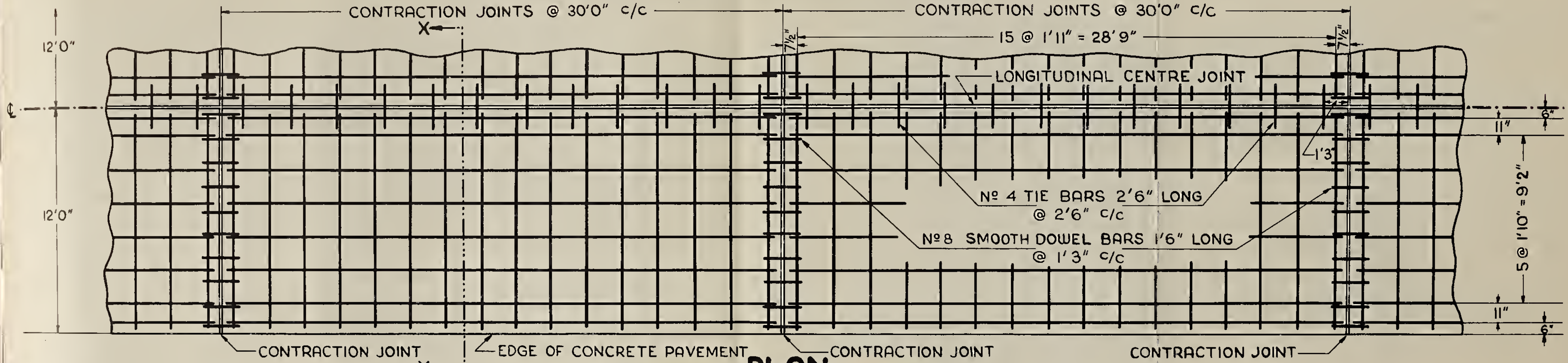
1. Concrete Pavement Sections
2. Reinforced Concrete Pavement Type "A"
3. Continuously Reinforced Concrete Pavement Type "B"
4. Non-Reinforced Concrete Pavement Type "C"
5. Reinforced Concrete Bridge Approach Slab
6. Transverse and Longitudinal Joint Details
7. Details of Expansion Dam
8. Details of Expansion Dam Assembly
9. Plan to Accompany Specifications (lists all the quantities, pavement types, etc.)



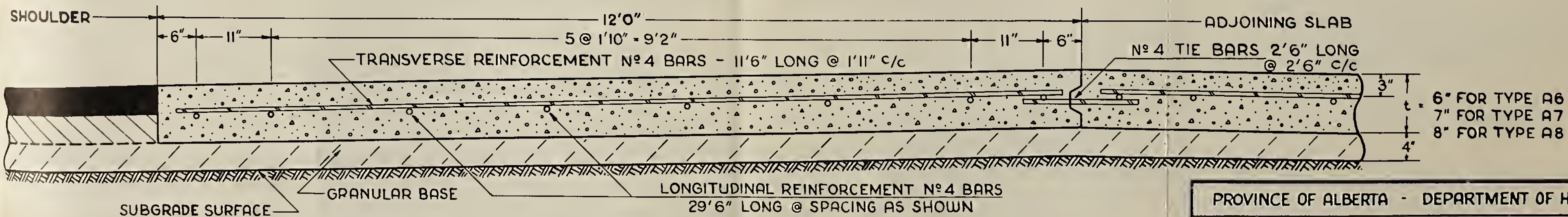
SUB BASE
CONTINUOUS 6" CONCRETE PAVEMENT - 0.7% LONG REINFORCING
#8 BARS - 14 BARS @ 10"
#4 BARS 11'-6" LONG @ 2'-6" O.C.
GRAVEL BASE
#6 BARS
MANITOBA REINFORCING DESIGN
0.12% LONG REINFORCING
CONCRETE PAVEMENT - MANITOBA REINFORCING DESIGN
0.16% LONG REINFORCING
CONCRETE PAVEMENT
PLAN Nº S 123
CONCRETE PAVEMENT
SECTIONS



TYPICAL CROSS-SECTION
FOR TYPE "A" REINFORCED CONCRETE PAVEMENT WITH ASPHALTIC BOUND SHOULDERS



PLAN
TYPICAL REINFORCED SLAB TYPE "A"



TRANSVERSE SECTION X - X

NOTE: LONGITUDINAL AND TRANSVERSE REINFORCING SHALL BE DEFORMED BARS OF HARD GRADE STEEL. DOWELS SHALL BE SMOOTH BARS OF HARD GRADE STEEL. TIE BARS SHALL BE DEFORMED BARS OF INTERMEDIATE OR STRUCTURAL GRADE STEEL.
FOR REINFORCED CONCRETE PAVEMENT TYPE A6 t = 6 INCHES, TYPE A7 t = 7 INCHES AND TYPE A8 t = 8 INCHES. SPACING AND SIZE OF LONGITUDINAL AND TRANSVERSE REINFORCING STEEL AS DETAILED SHALL BE THE SAME FOR TYPES A6, A7 AND A8 REINFORCED CONCRETE PAVEMENTS.

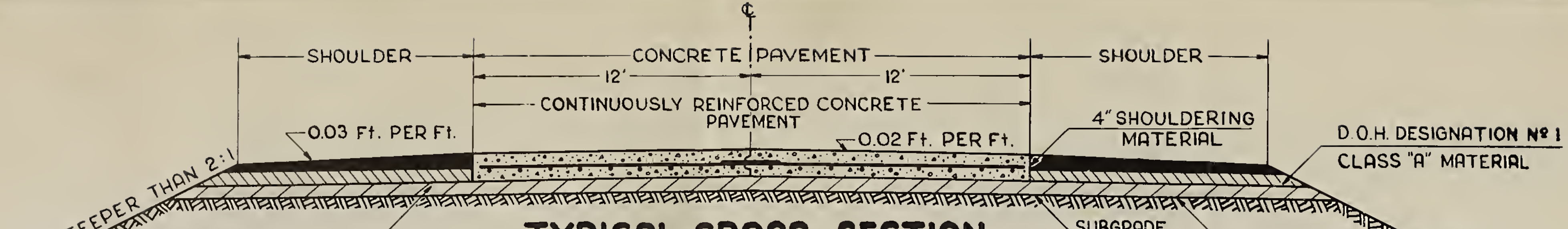
PROVINCE OF ALBERTA - DEPARTMENT OF HIGHWAYS

REINFORCED CONCRETE PAVEMENT TYPE "A"
2 LANES WITH ASPHALTIC BOUND SHOULDERS

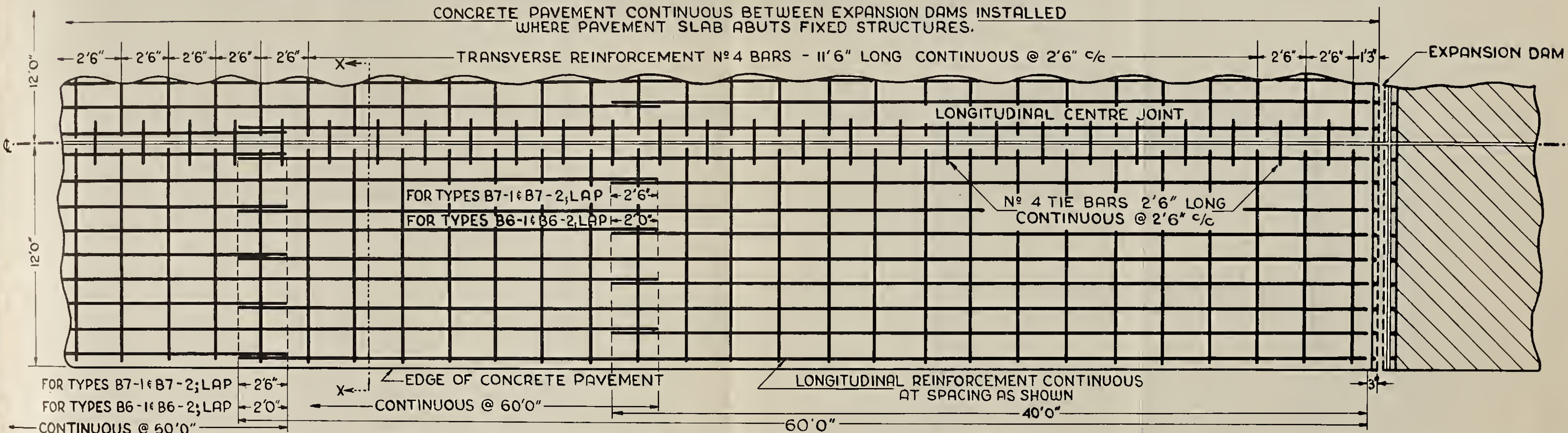
APPROVED

APRIL 22, 1958

CHIEF CONSTRUCTION ENGINEER

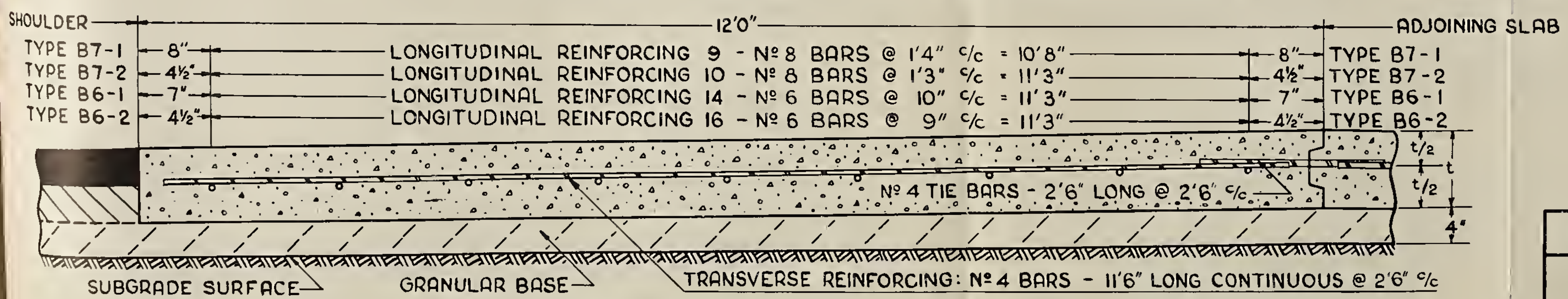


TYPICAL CROSS-SECTION
FOR TYPE "B" CONTINUOUSLY REINFORCED CONCRETE PAVEMENT WITH ASPHALTIC BOUND SHOULDERS



PLAN - TYPICAL CONTINUOUSLY REINFORCED SLAB TYPE "B"

NOTE: FOR TYPES B7-1 & B7-2 CONTINUOUSLY REINFORCED CONCRETE PAVEMENT: t=7 inches
FOR TYPES B6-1 & B6-2 CONTINUOUSLY REINFORCED CONCRETE PAVEMENT: t=6 inches



TRANSVERSE SECTION X - X

NOTE: LONGITUDINAL AND TRANSVERSE REINFORCING SHALL BE DEFORMED BARS OF HARD GRADE STEEL. TIE BARS SHALL BE DEFORMED BARS OF INTERMEDIATE OR STRUCTURAL GRADE STEEL.

SPACING AND SIZE OF TRANSVERSE REINFORCING STEEL SHALL BE THE SAME FOR TYPES B7-1, B7-2, B6-1 AND B6-2 CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS. SPACING AND SIZE OF LONGITUDINAL REINFORCING SHALL BE AS SHOWN ABOVE.

PROVINCE OF ALBERTA - DEPARTMENT OF HIGHWAYS

CONTINUOUSLY REINFORCED CONCRETE PAVEMENT TYPE "B"
2 LANES WITH ASPHALTIC BOUND SHOULDERS

APPROVED

APRIL 22, 1958

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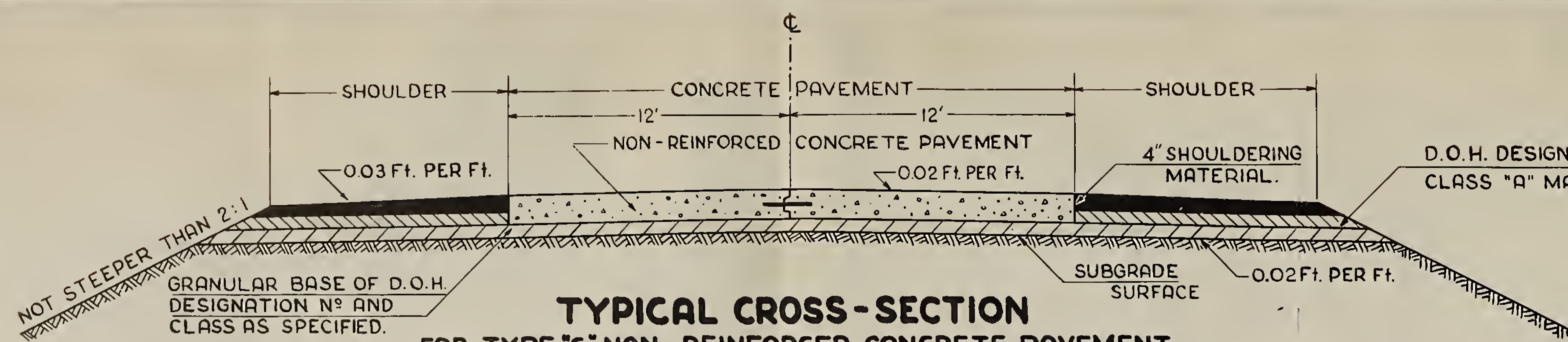
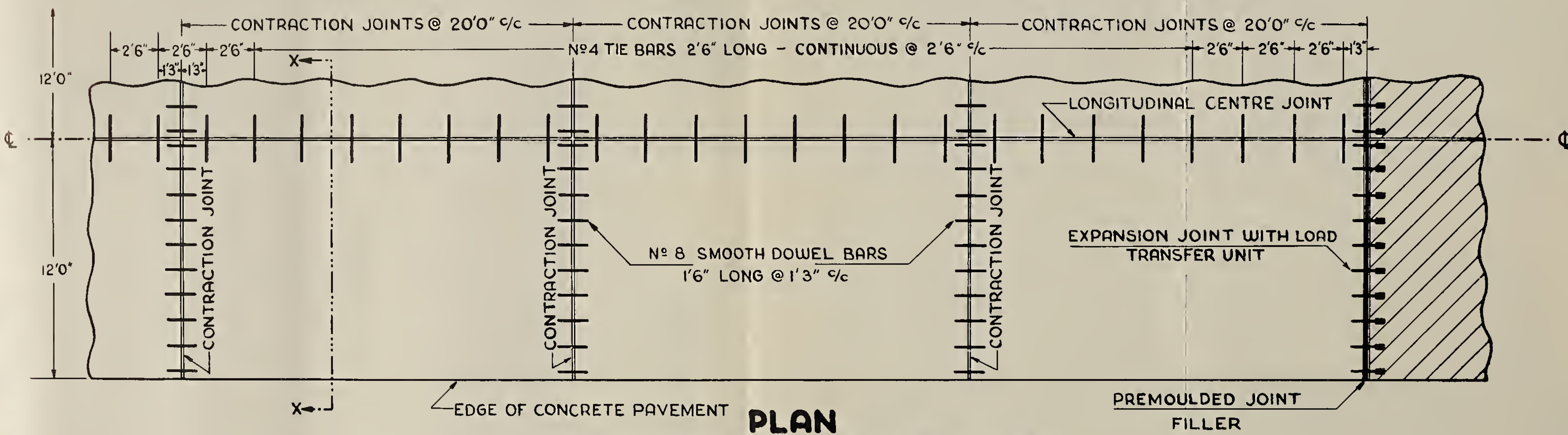


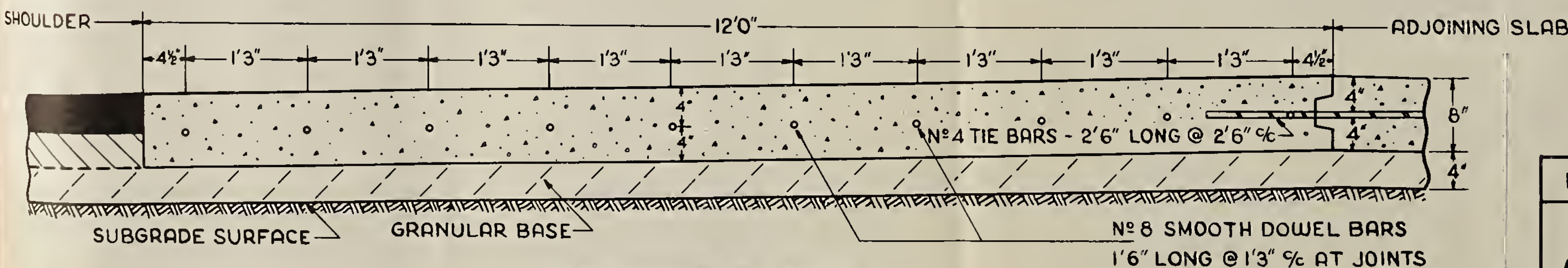
TYPICAL CROSS-SECTION

FOR TYPE "C" NON-REINFORCED CONCRETE PAVEMENT WITH ASPHALTIC BOUND SHOULDERS



PLAN

TYPICAL NON-REINFORCED SLAB TYPE "C"



TRANSVERSE SECTION X-X

NOTE: TIE BARS SHALL BE DEFORMED BARS OF INTERMEDIATE OR STRUCTURAL GRADE STEEL.
DOWEL BARS SHALL BE SMOOTH BARS OF HARD GRADE STEEL.

PROVINCE OF ALBERTA - DEPARTMENT OF HIGHWAYS

NON-REINFORCED CONCRETE PAVEMENT TYPE "C"

2 LANES WITH ASPHALTIC BOUND SHOULDERS

APPROVED

APRIL 22, 1958

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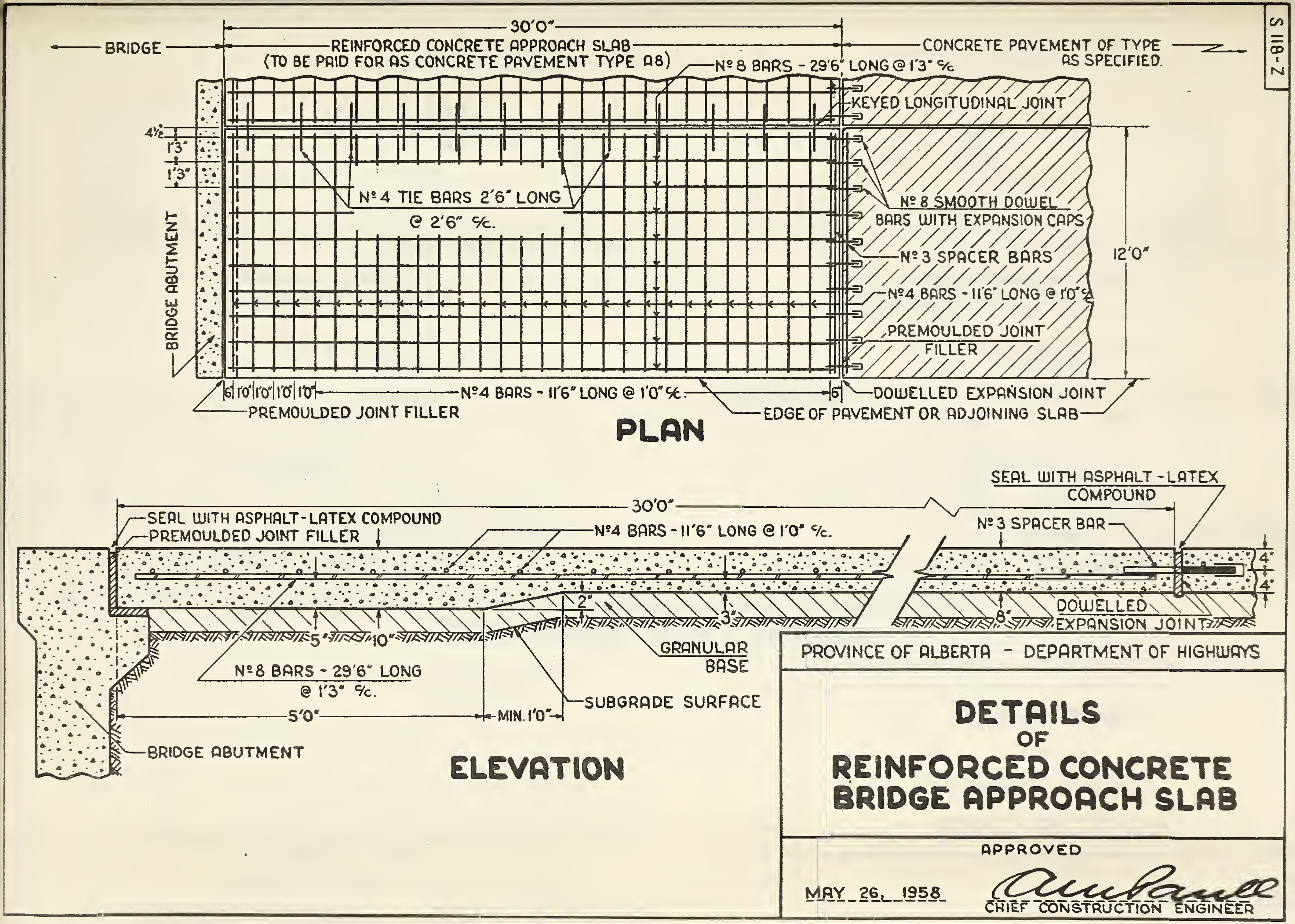
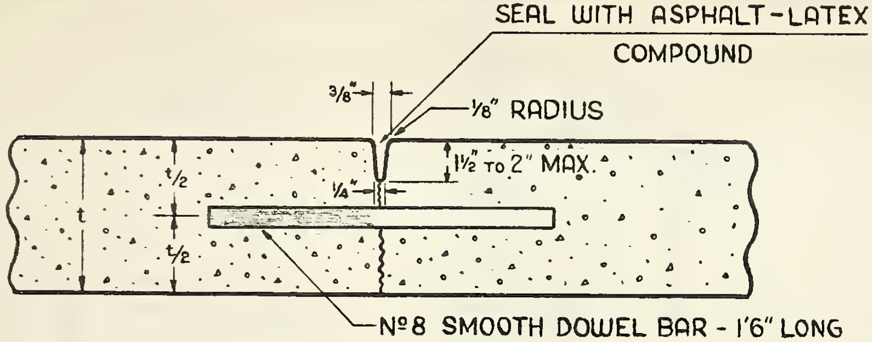
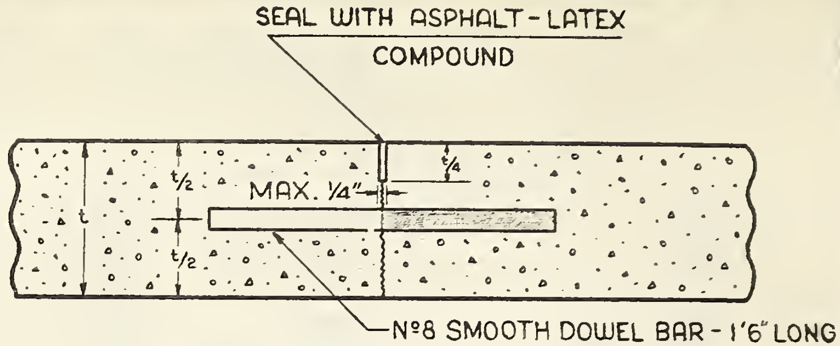

S 118-Z
BRIDGE
30'0"
REINFORCED CONCRETE APPROACH SLAB
(TO BE PAID FOR AS CONCRETE PAVEMENT TYPE A8)
CONCRETE PAVEMENT OF TYPE AS SPECIFIED.
Nº 8 BARS - 29'6" LONG @ 1'3" c/c
KEYED LONGITUDINAL JOINT
4½"
1'3"
1'3"
BRIDGE ABUTMENT
Nº 4 TIE BARS 2'6" LONG @ 2'6" c/c.
Nº 8 SMOOTH DOWEL BARS WITH EXPANSION CAPS
Nº 3 SPACER BARS
12'0"
Nº 4 BARS - 11'6" LONG @ 1'0" c/c
PREMOULDED JOINT FILLER
6 | 1'0" | 1'0" | 1'0" | 1'0"
Nº 4 BARS - 11'6" LONG @ 1'0" c/c.
6
DOWELLED EXPANSION JOINT
PREMOULDED JOINT FILLER
EDGE OF PAVEMENT OR ADJOINING SLAB
PLAN
SEAL WITH ASPHALT - LATEX COMPOUND
30'0"
SEAL WITH ASPHALT-LATEX COMPOUND
PREMOULDED JOINT FILLER
Nº 4 BARS - 11'6" LONG @ 1'0" c/c.
Nº 3 SPACER BAR
4"
4"
2"
3"
8"
DOWELLED EXPANSION JOINT
5"
10"
GRANULAR BASE
Nº 8 BARS - 29'6" LONG @ 1'3" c/c.
SUBGRADE SURFACE
5'0"
MIN. 1'0"
BRIDGE ABUTMENT
ELEVATION
PROVINCE OF ALBERTA - DEPARTMENT OF HIGHWAYS
DETAILS OF REINFORCED CONCRETE BRIDGE APPROACH SLAB
APPROVED
MAY 26, 1958
CHIEF CONSTRUCTION ENGINEER

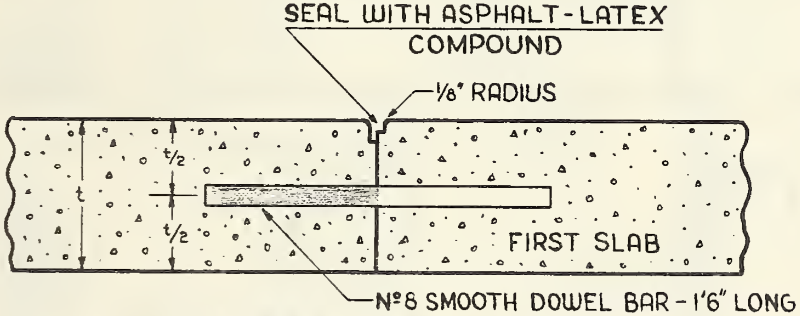
S 118-U



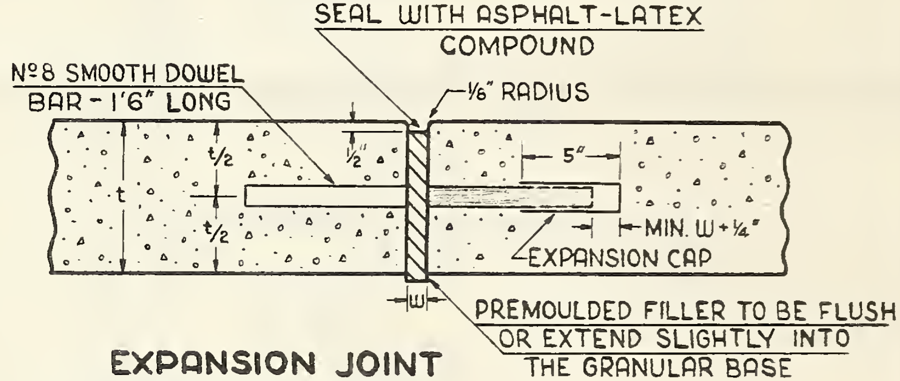
DUMMY GROOVE CONTRACTION JOINT



SAWED WEAKENED PLANE CONTRACTION JOINT



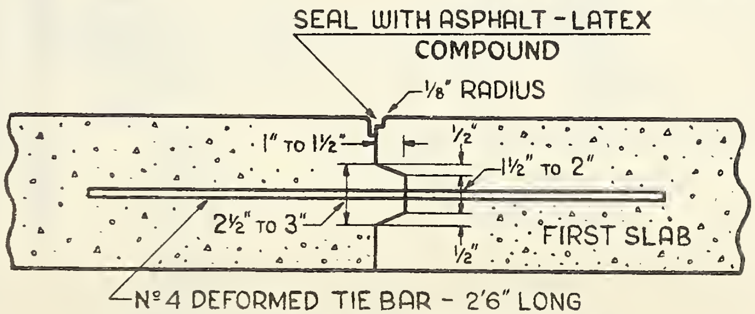
CONSTRUCTION JOINT



EXPANSION JOINT

NOTE: ONE HALF OF EACH DOWEL BAR ACROSS TRANSVERSE JOINTS SHALL BE LUBRICATED TO PREVENT BOND WITH CONCRETE. DOWEL BARS ACROSS EXPANSION JOINTS SHALL BE PROVIDED WITH EXPANSION CAPS.

TRANSVERSE JOINT DETAILS



KEYED LONGITUDINAL JOINT

LONGITUDINAL JOINT DETAILS

PROVINCE OF ALBERTA - DEPARTMENT OF HIGHWAYS

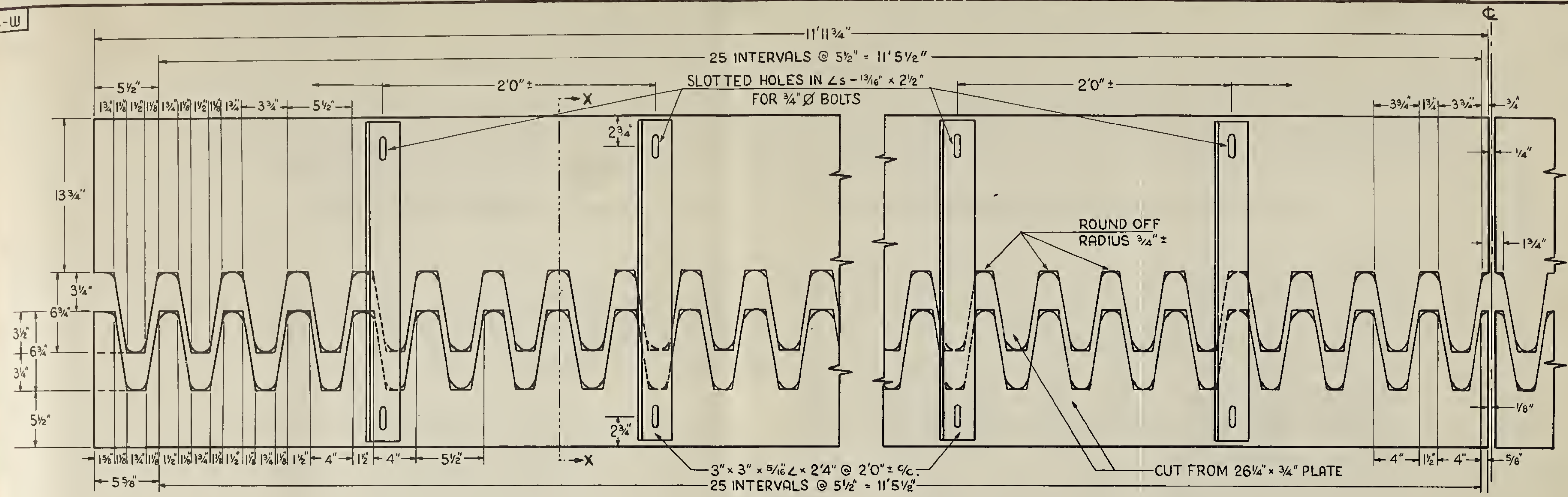
TRANSVERSE AND LONGITUDINAL JOINT DETAILS

APPROVED

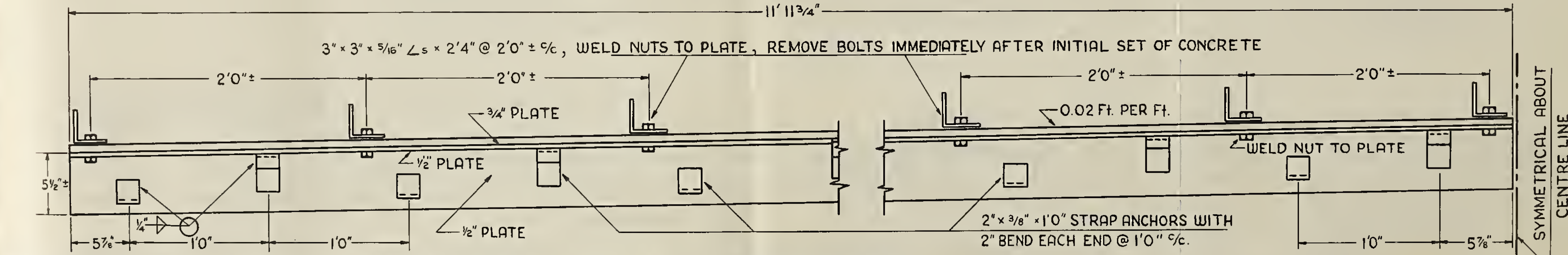
MAY 2, 1958

CHIEF CONSTRUCTION ENGINEER

S 118-W



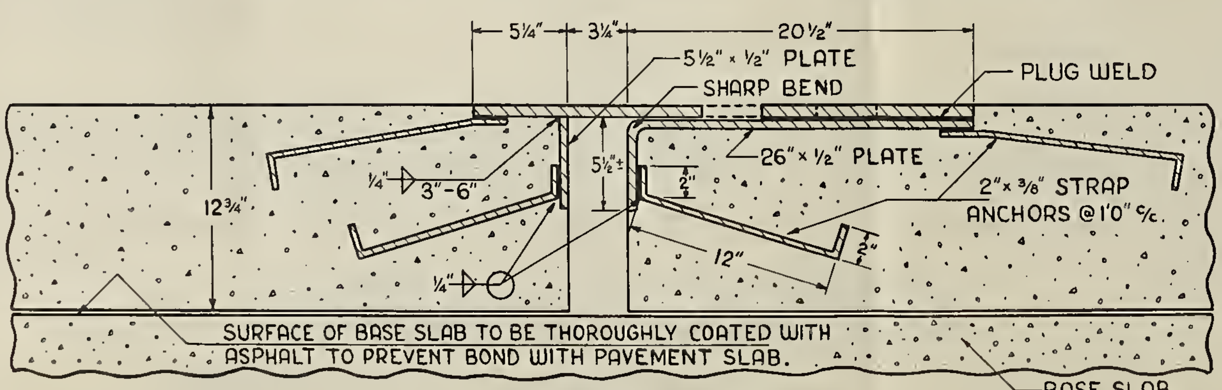
PLAN



NOTE: FOR EACH ASSEMBLY - 1 REQUIRED AS SHOWN AND 1 REQUIRED OPPOSITE HAND.

ELEVATION

NOTE: ALL SURFACES NOT IN CONTACT WITH CONCRETE OR SHOULDERING MATERIAL TO RECEIVE ONE SLAP - COAT OF RED LEAD PAINT.



SECTION X-X

PROVINCE OF ALBERTA - DEPARTMENT OF HIGHWAYS

DETAILS OF EXPANSION DAM

(TO BE USED WITH CONTINUOUSLY REINFORCED CONCRETE PAVEMENT TYPE "B")

APPROVED

MAY 26, 1958

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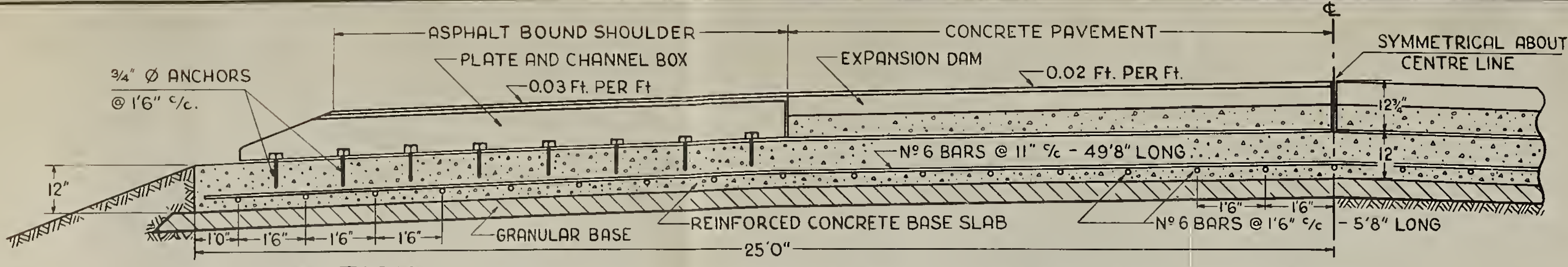
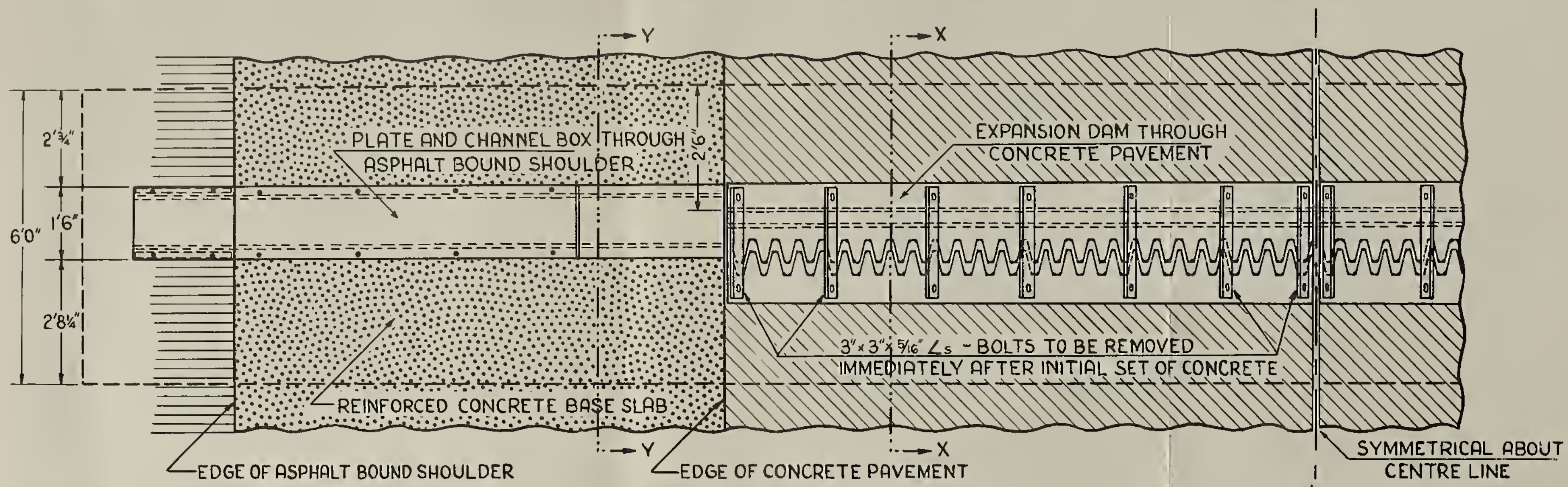
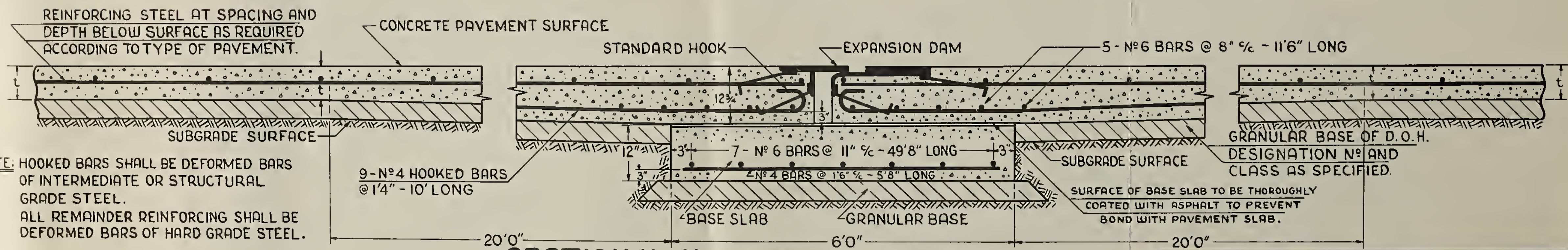


HALF SECTION – EXPANSION DAM WITH PLATE AND CHANNEL BOX THROUGH SHOULDER

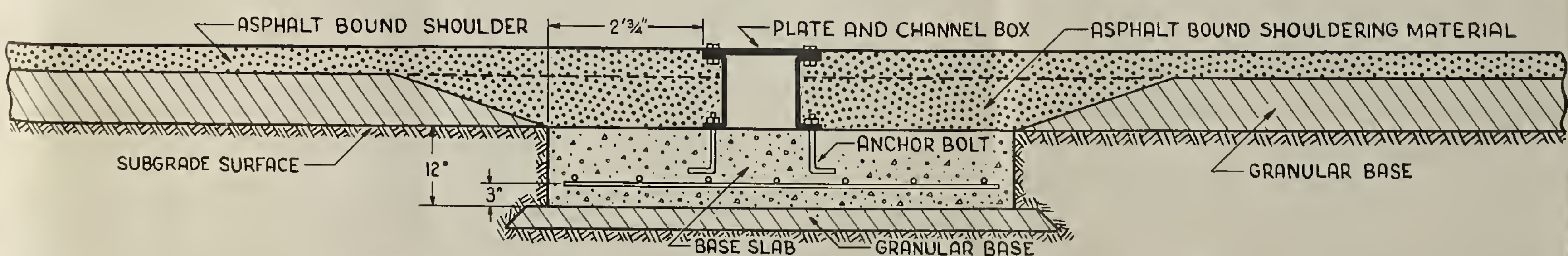


PLAN



NOTE: HOOKED BARS SHALL BE DEFORMED BARS OF INTERMEDIATE OR STRUCTURAL GRADE STEEL.
ALL REMAINDER REINFORCING SHALL BE DEFORMED BARS OF HARD GRADE STEEL.

SECTION X – X – EXPANSION DAM IN PLACE



SECTION Y – Y – PLATE AND CHANNEL BOX IN PLACE

PROVINCE OF ALBERTA - DEPARTMENT OF HIGHWAYS

DETAILS OF EXPANSION DAM ASSEMBLY

APPROVED

MAY 26, 1958

CHIEF CONSTRUCTION ENGINEER

PORTLAND CEMENT CONCRETE

PREPARATION OF SUBGRADE

ITEM	
PREPARING SUBGRADE SURFACE AREA	141,560

GRANULAR BASE

GRAVEL TO BE CRUSHED FROM PIT RUN STOCKPILE IN NW ¼ 26 - 24 - 2 - 5

FINISHED WIDTH	FOR WIDTHS SEE "DETAIL" PLAN BELOW				
FINISHED THICKNESS	4" BELOW CONCRETE & 8" BELOW SHOULDERING	4" BELOW CONCRETE & 7" BELOW SHOULDERING	4" BELOW CONCRETE & 6" BELOW SHOULDERING	4" BELOW CONC. & 7" BELOW SHOULD.	4" BELOW CO & 8" BELOW SHO
D.O.H. DESIGNATION Nº 1 CLASS "A" MATERIAL	9,016	8,535	9,919	4,986	7,175
HAUL	36,425	26,011	18,881	4,986	7,175
	0 - 5 MILES				

PORTLAND CEMENT CONCRETE

FINISHED WIDTH	24 FEET							
FINISHED THICKNESS	8 INCHES	7 INCHES		6 INCHES			7 INCHES	8 INCHE
PORTLAND CEMENT CONCRETE TYPE A6						6,960		
" " " TYPE A7							8,240	
" " " TYPE A8	53							9,440
" " " TYPE B6-1				5,259				
" " " TYPE B6-2					5,879			
" " " TYPE B7-1			7,053					
" " " TYPE B7-2		7,061						
" " " TYPE C	13,918							
REINFORCING STEEL	25,251	166,590	150,668	99,234	125,777	57,450	68,109	80:920
EXPANSION DAM ASSEMBLY		1			1			
TACK COAT AREA	61,400							
PRIME COAT AREA	61,400							

FINISHED WIDTH	
FINISHED THICKNESS	
D.O.H. DESIGNATION Nº 1 CLASS "A" MATERIAL	
HAUL	

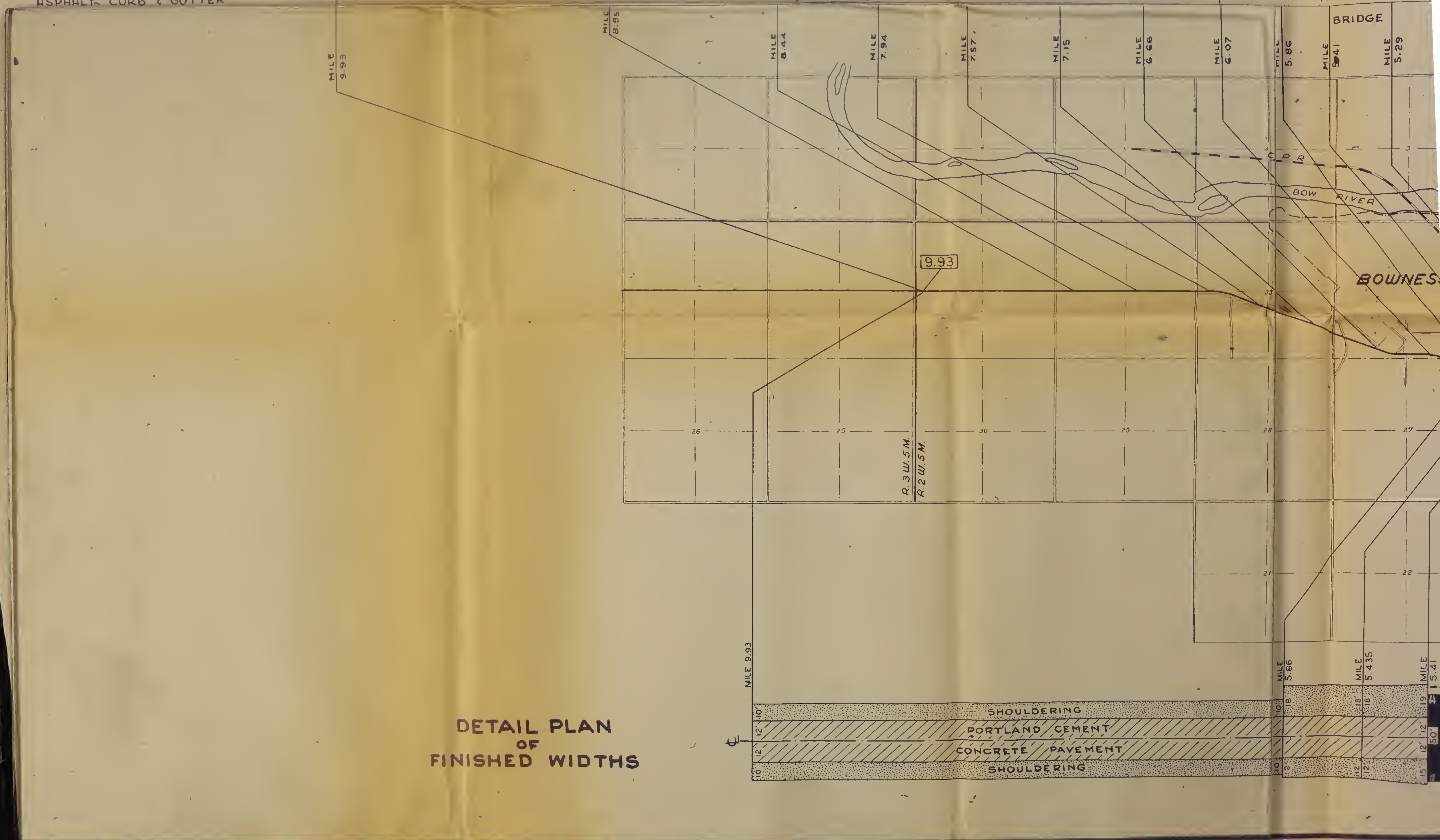
SHOULDERING

GRAVEL TO BE CRUSHED FROM PIT RUN STOCKPILE IN NW ¼ 26 - 24 - 2 - 5

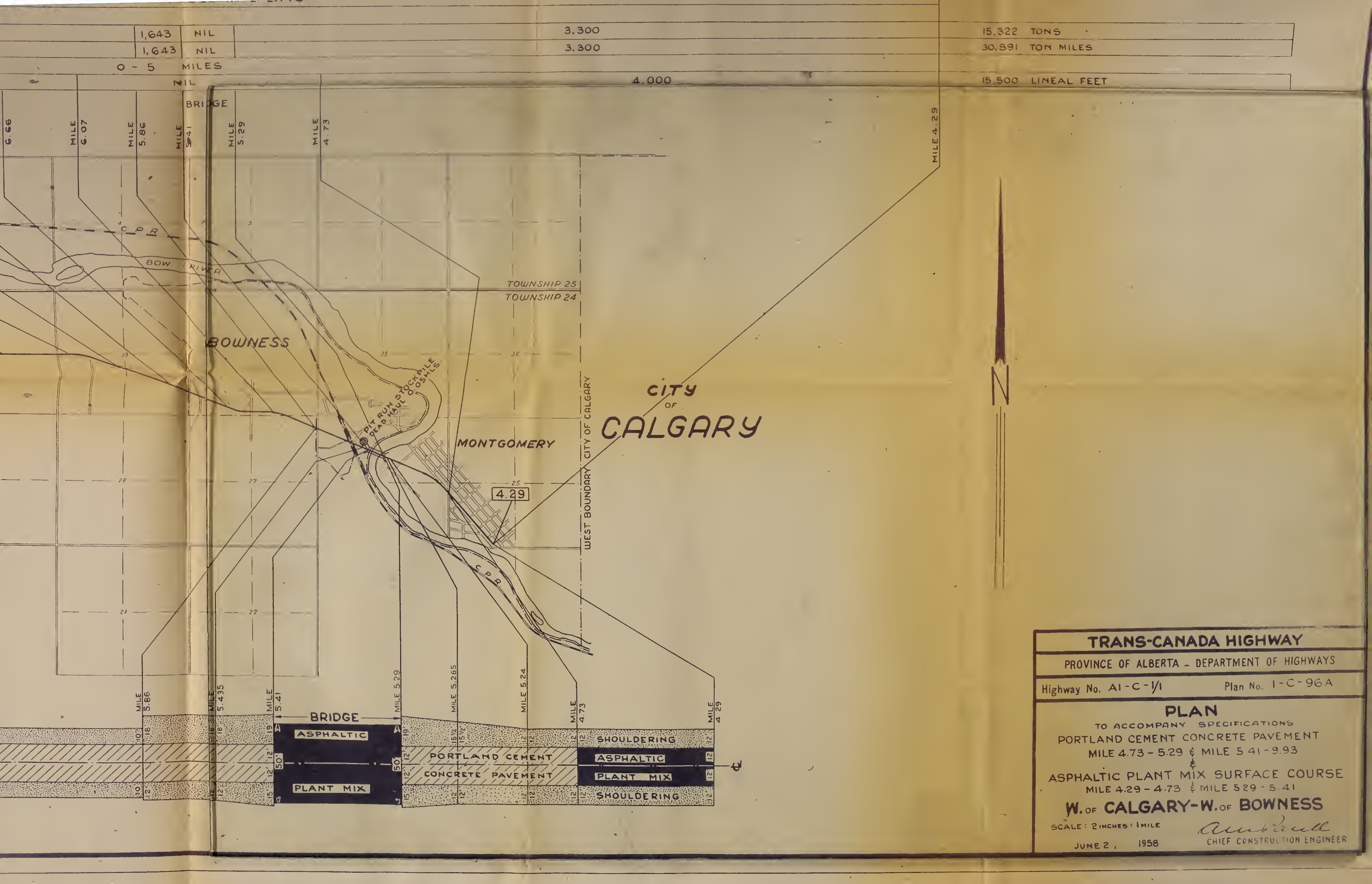
FINISHED WIDTH	FOR WIDTHS SEE "DETAIL" BELOW
FINISHED THICKNESS	THICKNESS 4 INCHES TAPERED TO 3" AT SHOULDER

		BRIDGE		ASPHALTIC PLANT MIX SURF. COURSE	
					TOTALS
		BRIDGE	18, 440		160,000 SQUARE YARDS
ASE					
		BRIDGE		GRAVEL TO BE CRUSHED FROM PIT RUN STOCKPILE IN NW 1/4 26 - 24 - 2 - 5	
PLAN BELOW					
" BELOW CONC. & " BELOW SHOULD.	4" BELOW CONC. & 8" BELOW SHOULD.		4" BELOW CONC & 8" BELOW SHOULD.	SHOULDER RECONDITIONING ADJACENT TO EXISTING "PLANT MIX"	
4,986	7, 175	BRIDGE	5, 712	1, 584	46,927 TONS
4,986	7, 175		5, 712	1, 584	100, 774 TON MILES
		BRIDGE	24 FEET		
7 INCHES	8 INCHES		8 INCHES		
					6,960 SQUARE YARDS TYPE A - 6
8, 240					8, 240 SQUARE YARDS TYPE A - 7
	9, 440		8, 320		17, 813 SQUARE YARDS TYPE A - 8
					11, 138 SQUARE YARDS TYPE B - 6
					14, 114 SQUARE YARDS TYPE B - 7
					13, 918 SQUARE YARDS TYPE C
68,109	80,920	BRIDGE	71, 581		845,580 POUNDS REINFORCING STEEL
					2 EXPANSION DAM ASSEMBLIES
		,500	8,600	26,500	170, 000 SQUARE YARDS
		BRIDGE	8,600		
	0 - 5	MILES			
				ASPHALTIC PLANT MIX SURFACE COURSE	
		50 FT.		24 FEET	
		1 LIFT OF 2 INCHES		4 INCHES (2 LIFTS OF 2" EACH)	
				GRAVEL TO BE CRUSHED FROM PIT RUN STOCKPILE IN NW 1/4 26 - 24 - 2 - 5	
		370		2, 599	2,969 TONS
		370		2, 599	2, 969 TON MILES
		0 - 5		0 - 5 MILES	
OULDERING					
		BRIDGE		GRAVEL TO BE CRUSHED FROM PIT RUN STOCKPILE IN NW 1/4 26-24-2-5	
AIL" BELOW					
ED TO 3" AT SHOULDER EDGE IN 2 LIFTS					
	1,643	NIL		3,300	15,322 TONS

FINISHED THICKNESS			
D.O.H DESIGNATION Nº 1 CLASS "A" MATERIAL	10,379	1,643	NIL
HAUL	25,948	1,643	NIL
		0 - 5 MILES	
ASPHALT CURB & GUTTER	11,500	NIL	



DETAIL PLAN
OF
FINISHED WIDTHS



1,643 NIL 3,300 15,322 TONS
1,643 NIL 3,300 30,891 TON MILES
0 - 5 MILES
NIL 4.000 15,500 LINEAL FEET
BRIDGE
MILE 6.07
MILE 5.86
MILE 5.41
MILE 5.29
MILE 4.73
MILE 4.29
C P R
BOW RIVER
TOWNSHIP 25
TOWNSHIP 24
BOWNESS
PIT RUN STOCKPILE
DEAD HAUL 0.05 MLS.
MONTGOMERY
WEST BOUNDARY CITY OF CALGARY
CITY OF CALGARY
4.29
N
MILE 5.86
MILE 5.435
MILE 5.41
MILE 5.29
MILE 5.265
MILE 5.24
MILE 4.73
MILE 4.29
BRIDGE
ASPHALTIC
PLANT MIX
SHOULDERING
PORTLAND CEMENT
CONCRETE PAVEMENT
ASPHALTIC
PLANT MIX
SHOULDERING
TRANS-CANADA HIGHWAY
PROVINCE OF ALBERTA - DEPARTMENT OF HIGHWAYS
Highway No. A1-C-1/1
Plan No. 1-C-96A
PLAN
TO ACCOMPANY SPECIFICATIONS
PORTLAND CEMENT CONCRETE PAVEMENT
MILE 4.73 - 5.29 & MILE 5.41 - 9.93
&
ASPHALTIC PLANT MIX SURFACE COURSE
MILE 4.29 - 4.73 & MILE 5.29 - 5.41
W. OF CALGARY - W. OF BOWNESS
SCALE: 2 INCHES : 1 MILE
JUNE 2, 1958
CHIEF CONSTRUCTION ENGINEER

A P P E N D I X   B

SUMMARY OF CONCRETE STRENGTH DATA FROM SAMPLES TAKEN DURING CONSTRUCTION

# 1 HIGHWAY	Slump	% Air	10 Day Flexural Strength	28 Day Flexural Strength	28 Day Cylinder Compressive Strength
158 + 50				630	
158 + 75	1	4.1	550 580	630	5000 4620
162 + 50	1¼	4.3	590		
166 + 80				560	
167 + 25	2	6.8	490 500	820	4010 3280
171 + 00	2¼	5.7	570 620	620 720	3120 3370
175 + 00	1 3/4	5.2	620 600	690	3620 3630
179 + 25	1½	3.7	640 570	630	4320 4410
188 + 50	1 3/4	2.6		660	
189 + 00	1 3/4	6.0	570 520	640	2930 3600
	1½	7.0	570		
191 + 85	1½	7.0	600	820	4150 4220
196 + 50	1¼	5.9	540 550	770 680	4310 4530
199 + 50	1¼	4.4	610 550	670 630	4950 5510
205 + 00				500	
205 + 25	2¼	2.1	600 580	740	5540 5680

Note: The units of strength are p.s.i., and those of slump are inches.

	Slump	% Air	10 Day Flexural Strength	28 Day Flexural Strength	28 Day Cylinder Compressive Strength
304 + 50	1	2.0	660 590	760	4810 4220
310 + 40	1	3.4	690 710	890	5290 5310
316 + 70	2	4.8	610 625	730	4290 4300
318 = 35	3/4	3.8	630 635	675	
325 + 50	$1\frac{1}{4}$	4.5	595 695	785	4850 5200
329 + 60	$1\frac{1}{2}$	6.6	685 625	805	4340 4450
334 + 60	1 3/4	1.8	540 550	705	5270 5340
342 + 90	$1\frac{1}{2}$	4.1	670 725	865	4410 4540
350 + 60	1 3/4	4.6	745 870	915	4930 4810
355 + 50	1	3.6	715 810	805	5170 5350
360 + 30	$1\frac{1}{4}$	4.2	745 805	780	5480 5510
369 + 80	2	4.2	650 695	695	5560 5320
373 + 20					5240 4760
375 + 60	1	1.5		710	5460 5240
377 + 60	$\frac{1}{2}$	2.0		765	5050 5410
380 + 00	$1\frac{1}{2}$	2.0		890	5110 6120

	Slump	% Air	10 Day Flexural Strength	28 Day Flexural Strength	28 Day Cylinder Compressive Strength
210 + 00	1¼	4.1	560 630	690 650	5170 5410
216 + 00	2½	4.5		640 600	4070 3950
233 + 50	1¼	2.8		890 670	5690 5116
236 + 40	2¼	5.8		690	3930 3820
237 + 50				700	
243 + 24	1½	2.6		730 400	5630 5920
248 + 00	1½	5.0		670	4600 4350
253 + 00	1	3.2		790	6270 5826
254 + 80	1¼	5.0	580 620	595	3500 3650
259 + 60	2	6.2	440	530	3440 3570
265 + 00	1¼	3.6	600 570	580	4520 4460
270 + 60	1½	2.4	700 650	700	5380
275 + 25	1 3/4	3.2	555 575	665	4890 4820
281 + 30	1½	4.0	500 465	590	
286 + 00	3	4.0	500	495	3990 3960
290 + 60	2	4.6	490 545	655	4070 3370
299 + 45	1	1.8	575 630	570	4960 4990

	Slump	% Air	10 Day Flexural Strength	28 Day Flexural Strength	28 Day Cylinder Compressive Strength
383 + 20	1	4.5	665 635	715	4200 4180
386 + 00	2	3.2	670 650	780	4880 4620
388 + 00	1 3/4	4.5	615 570	730	3160 3030
391 + 25	1	5.4	560 605	630	4930 4690
395 + 50	3	3.9	600 590	745	5380 5196
398 + 50	1	3.8	645 m620	675	3810 5610
401 + 50	$1\frac{1}{4}$	4.3	670 650	875	5680 5830
405 + 20	1 3/4	5.4	610 620	725	5430
406 + 70	$1\frac{1}{4}$	4.3	580 640	755	4870
408 + 50	2	5.5	555 680	745	4940
410 + 40	2	5.2	620 605		5170
412 + 80	1 3/4	4.5	630 620	700	5430
415 + 50	1 3/4	4.6	620 680	655	5470
419 + 00	$2\frac{1}{4}$	6.1	500 530	680	4130
419 + 50	$1\frac{1}{2}$	4.2	660 665	780	5510
421 + 20	$\frac{1}{2}$	4.0	695	760	5480
422 + 60	$1\frac{1}{2}$	6.0	685 705		

	Slump	% Air	10 Day Flexural Strength	28 Day Flexural Strength	28 Day Cylinder Compressive Strength
423 + 20	1	2.2	780 830		
425 + 75	½	4.9	585 725		

2 HIGHWAY

East Outer Lane

	Slump	10 Day Flexural Strength
111 + 00	2¼	460
118 + 00	½	620
124 + 00	1	600
130 + 00	1¼	520
141 + 00	2	525
143 + 50	1	675
150 + 00	1 3/4	530

East Inner Lane

	Slump	10 Day Flexural Strength
109 + 50	2 3/4	570
111 + 50	3	520
115 + 00	1 3/4	570
118 + 80	2	460
122 + 00	2¼	450
125 + 50	2	410
130 + 00	2½	490
133 + 00	1 3/4	590
136 + 00	2	460
140 + 00	1 3/4	520
144 + 50	2¼	500
150 + 00	1 3/4	520

West Outer Lane

102 + 00	1 3/4	515
114 + 00	1	540
123 + 00	1½	615
130 + 00	1½	635

West Inner Lane

108 + 00	1½	390
110 + 50	2½	340
115 + 00	2½	290
120 + 60	1 3/4	380

West Outer Lane (cont)

	Slump	10 Day Flexural Strength
136 + 50	1½	560
144 + 50	1	560
150 + 50	1½	540

West Inner Lane (cont)

	Slump	10 Day Flexural Strength
127 + 90	3/4	630
133 + 00	1¼	530
141 + 00	1 3/4	520
147 + 00	1½	560
152 + 50	1 3/4	600

APPENDIX C

ESTIMATION OF PROBABLE MAXIMUM JOINT WIDTH AT ANY PARTICULAR TEMPERATURE FOR FUTURE DESIGN PURPOSES

The problem consists essentially of estimating the average crack width by utilizing data in Table 15-1. Then from data in Chapter XII the ratio "wide crack width to average crack width" can be determined, and hence the probable width of the widest joint crack at any particular temperature can be estimated.

The average crack width is given by:

$$X = (To - T) \propto Ls$$

From Table 15-1 a suitable value for $\propto$, for the aggregate and mix used on #1 Highway would be about 6.0×10^{-6} inches/inch/°F. To can be estimated by estimating the pouring temperature Tp of the mix, and adding 10°F to allow for curing shrinkage (t_s) and 10°F for creep (t_c) in the reinforced slabs, and 20°F in the plain slabs. Thus, if Tp was estimated at 60°F, then To for the reinforced slabs would be taken as 80°F, and To for the plain slabs as 90°F.

The pouring temperature can be estimated by considering the average daily temperature at the time of pouring. The pouring temperature of the mix will be roughly the temperature of the aggregate within the stockpile, and this roughly represents the average temperature of the daily period at that time.

Knowing the length of the slabs, the average joint crack width at the temperature T (lower than To) can be estimated.

The maximum joint crack width at this temperature can be estimated from:

$$Ww = R.X$$

Chapter XII shows that R = 3 during the first winter in the type C pavements, but later became 1.9. With pavement type A-8, at location A-8-6, R was originally 2.4, but later reduced to 1.3.

If no particular care is to be taken in dowel setting, then R should be taken as about 2.0 for long term estimates. If more care in dowel setting is taken, a value of 1.2 might be more suitable.

The use of the ratio R is only necessary for estimates concerning shorter slabs with doweled joints; i.e. similar to the designs used on #1 Highway. With no dowels in the joints, or with slab lengths of 100 feet, the use of the R ratio is unnecessary and the actual joint width can probably be taken as the average joint width.

APPENDIX D

DESIGN - USING RECOMMENDATIONS GIVEN BY A.C.I. COMMITTEE 325. A.C.I. PAPER 53-59 COMPARISON WITH DESIGN USED ON PROJECT.

Design Data	Subgrade k value = 170 lbs. per cubic inch Maximum legal axle load - 18,000 lbs. Allow 2,000 lbs. additional Allow 10% for impact, hence design load = 22,000 lbs.
Sub-Base	To prevent pumping - 3" to 6" Used on project - 4" to 8"
Slab Length	Unreinforced - 20' recommended Used on project - 20' Reinforced - 40' to 100' recommended, depending on quantity of reinforcement. Used on project - 30'
Slab Thickness	Charts A401 (a) and (b) Unreinforced:(design for unprotected corners)-8" Used on project - 8" Reinforced with load transfer device:(design for protected corners) - $7\frac{1}{4}$" Used on project - 6", 7" and 8"
Expansion Joints	Depend upon local conditions. However, should use at bridges, etc. Used on project - adjacent to each bridge abutment - one 1" expansion joint.
Contraction Joint	Groove Type Depth of groove not less than 1/6 D. not greater than 1/4 D. Used on project - 1/4 D. Width - at least twice the anticipated annual variation in joint width, but not less than 1/8" Used on project - 1/8"
Dowel Length and Spacing	Mechanical load transfer devices are recommended when joint spacing exceeds 20', and even at a spacing of 20' if loading conditions are severe.

Slab 6" Dowel Diam	3/4"	Length 18"	Spacing 12"
7"	1"	18"	12"
8"	1"	18"	12"

Used on project for all slab thicknesses

8" plain concrete 20' slab	1"	15"	15"
6" 7" and 8" reinforced 30' slab	1"	15"	15"

Slab length 20' plain - no mechanical load transfer device

Longitudinal Reinforcement

$$As = \frac{F.L.W.}{2f_s} \text{ square inches per foot width}$$

F = coefficient of subgrade friction = 1.5
L = slab length in feet = 30
W = weight of slab in p.s.f. = 96 for 8" slab
72 for 6" slab
F_s = working stress, 2/3 yield stress = 36,000 p.s.i.
(yield stress of 55,000 p.s.i. obtained by Bereznicki)

Hence for 6" slab As = 0.045 sq. inches per foot width
8" slab As = 0.06

Used on project 0.12 sq. inches per foot width, for 6", 7" and 8" slab. The reinforcement used was #4 bars at 22" spacing. (The recommended spacing is not greater than 15")

Tie Bars

Working Stress 33,000   12' lane   #4 bars

Pavement thickness 6"	length 27"	spacing 48"
7"	27"	48"
8"	27"	43"

Used on project

6", 7", 8" Reinforced Slab) 8" Plain Slab)	30"	30"

8" Plain slab - no tie bars

Transverse Reinforcement

Equal to the area of steel used in the tie bars. Used on the project - #4 bars at 23" centres.